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PROGRESSIVE FAILURE IN OVERCONSOLIDATED CLAYS AND MUDSTONES

by

(C) KURT DIETER EIGENBROD

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

AND RESEARCH IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

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EDMONTON, ALBERTA

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ABSTRACT

A review of previous work on progressive failure in overconsolidated clays and mudstones indicated many misinterpretations and complications of this problem. In order to avoid further misunderstandings progressive failure was defined in the following manner:

"Progressive failure exists in the field, when on average the strength mobilized at failure is less than the peak strength exhibited by testing a representative element of the slope in a representative manner."

It was shown that the basic distinction between mechanisms causing fully softened strength conditions and mechanisms causing residual strength conditions is arbitrary. The two strength conditions represent just two successive stages of strength reduction. During the first stage of strength reduction the cohesion intercept is reduced as a result of dilatancy. Fully softened strength conditions ($c' = 0$; $\phi' = \phi'_{peak}$) are the final result of this process. Subsequent large shear strains lead to the second stage of strength reduction. Continuous shear planes develop causing a reduction of the effective friction angle. Finally, after very large strains, residual strength conditions ($c' = 0$; $\phi' = \phi'_{residual}$) can be obtained.

Field evidence indicated that the average strength mobilized along a first-time failure plane is never less than the fully softened strength. Residual strength conditions were found only along reactivated shear planes after very large shear strains. For non-fissured materi-

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A review of previous work on progressive failure in overconsolidated clays and mudstones indicated many misinterpretations and complications of this problem. In order to avoid further misunderstandings progressive failure was defined in the following manner:

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It was shown that the basic distinction between mechanisms causing fully softened strength conditions and mechanisms causing residual strength conditions is arbitrary. The two strength conditions represent just two successive stages of strength reduction. During the first stage of strength reduction the cohesion intercept is reduced as a result of dilatancy. Fully softened strength conditions ($c' = 0$; $\phi' = \phi'_{peak}$) are the final result of this process. Subsequent large shear strains lead to the second stage of strength reduction. Continuous shear planes develop causing a reduction of the effective friction angle. Finally, after very large strains, residual strength conditions ($c' = 0$; $\phi' = \phi'_{residual}$) can be obtained.

Field evidence indicated that the average strength mobilized along a first-time failure plane is never less than the fully softened strength. Residual strength conditions were found only along reactivated shear planes after very large shear strains. For non-fissured materi-

als with flat-topped stress strain curves it was shown that prefailure strength reduction is unlikely to occur.

The importance of knowing the representative strength of a slope material was pointed out. The strength of a slope is governed by zones of weakness, which often are very small features and in many cases can be recognized only by knowing the factors which created them. Nonhomogeneous water pressures have to be expected in many slopes, as well. Therefore a special concern for minor geological details is always recommended.

Pore pressure changes upon excavation of a slope were found to be predictible in relatively homogeneous materials by a Finite Element Analysis. A numerical analysis of the dissipation of excess pore pressures due to unloading indicated that this process might possibly account for many delayed slope failures.

For overconsolidated clays and mudstones a classification was suggested on the basis of water deterioration tests. The standard compression softening test differentiates between clays and rocks according to strength reduction during water immersion of materials at their natural water contents. Clays were further classified according to rate of softening into hard, stiff, and medium to soft clays. Amount of strength softening can be predicted from simple compression test results. Clays and mudstones can also be described in terms of

slaking properties. Amount of slaking is predictible from the Liquid Limit of the materials. The rate of slaking can be obtained from a simple water immersion test of ovendried specimens.

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GLOSSARY OF SYMBOLS

A	= Activity = $\frac{I_p}{\%\ \text{Clay Fraction}}$
A	= Pore Pressure Coefficient
B	= Pore Pressure Coefficient
c'	= cohesion in terms of effective stresses
c'_{peak}	= peak cohesion values in terms of effective stresses
c_v	= Coefficient of Consolidation = $\frac{k}{m_v\ \gamma_w}$
c_s	= Coefficient of Swelling
c_u	= undrained shear strength = $\frac{q_u}{2}$
c_{uo}	= initial shear strength
c_{uf}	= fully softened shear strength
Δc_u	= full strength loss during softening = $c_{uo} - c_{uf}$
E	= undrained Modulus of Elasticity
E_o	= initial Modulus of Elasticity
E_f	= Modulus of Elasticity after full softening
ΔE	= change of Modulus of Elasticity during softening = $E_o - E_f$
ϵ	= unit strain
F_s	= Factor of Safety = $\frac{\text{available shear strength}}{\text{mobilized shear strength}}$
ϕ'	= angle of friction in terms of effective stresses
ϕ'_{peak}	= effective peak angle of friction

$\phi'_{residual}$ = effective residual angle of friction

γ_t = total unit weight of a soil

γ_w = unit weight of water = 62.4 lb/ft^3

H = height of embankment

h = depth below groundsurface

I_L = Liquidity Index = $\frac{w_n - w_p}{I_p}$

I_{LO} = Liquidity Index of undisturbed material

I_{LI} = Liquidity Index after the first slaking cycle

ΔI_L = change of Liquidity Index during the first slaking cycle = $I_{LI} - I_{LO} = \frac{w_I - w_n}{I_p}$

I_p = Plasticity Index = $w_L - w_p$

I_W = Work Softening Index

K_o = Coefficient of Earth Pressure at Rest

k = permeability (m/sec)

m_v = Coefficient of Volume Compressibility (cm^2/gm)

μ = micron

ν = Poisson's Ratio

q_u = unconfined compressive strength

σ = total stress

σ' = effective stress

σ_1, σ_3 = principal stresses

σ_v = vertical stress

σ_h = horizontal stress

σ_x, σ_y = stresses in x- and y- direction

$\Delta\sigma_1$, $\Delta\sigma_3$ = change of principal stresses

$\Delta\sigma_x$, $\Delta\sigma_y$ = change of stresses in x- and y- direction

t = time since inception of dissipation of excess pore pressures

t_s = time of softening during water immersion

τ = time factor = $c_v t/H^2$

u = pore pressure

u^* = pore pressure before excavation

u_o = pore pressure immediately after excavation = $u^* - \Delta u$

u_t = pore pressure at steady seapage conditions

Δu = change in pore pressures due to unloading = $u^* - u_o$

Δu_e = excess pore pressures = $u_o - u_t$

w = water content in percent of dry weight

w = equilibrium water content after each wetting stage

w_I = water content after 2 hours of water immersion

= water content after 1st cycle of slaking

w^* = maximum increase of water content during slaking

= $w_s - w_n$

w_L = Liquid Limit

w_n = natural water content

w_p = Plastic Limit

w_s = maximum water content due to slaking

Δw = increase of water content during softening

Δw_f = change of water content after full softening

CHAPTER I

REVIEW OF STUDIES ON PROGRESSIVE FAILURE

1. Historical Review

Slopes in overconsolidated clays and mudstones have always been very troublesome because of the occurrence of many failures which often happen some time after the slopes had been cut. A first explanation for this apparent decrease in strength with time was given by Terzaghi in 1936. He suggested that in "stiff fissured clays" the lateral stress release which results from excavating a cutting, could cause some opening of fissures and an increase of mass permeability. The strength of stiff clays is sufficient to keep a fissure open even at considerable depths and softening can then start from the face of the open fissures under zero effective stresses. This leads to a reduction in average strength, which allows more deformations to occur. More fissures then open up and the process continues. In 1948 Skempton continued along this line and inferred (although Terzaghi did not draw this conclusion) that,

> "unless slip intervenes, the end product of such a softening process must be a clay reduced essentially to its normally consolidated condition."

Quantitative data on the rate of softening in London Clay which were based on field evidence and undrained strengths at the time of failure

were quoted as an indication of a time dependent trend of strength reduction. Cassell (1948) compared laboratory strengths and field strengths in Lias, Oxford, and London Clays, and found that slips occurred in the field with the clay far softer than ever measured by routine laboratory tests. From field evidence shear strengths were estimated which were about an order of magnitude smaller than the laboratory strengths. The stable angles which were suggested for those formations were found to be in close agreement with the inclination of the natural slopes.

The early studies were generally limited to a $\phi = 0$ failure analysis. A more fundamental approach to the problem of slope stability became possible after the development of the effective stress analysis (Bishop, 1952 and 1955). The effective stress analysis based on reliable laboratory test data, provided a more dependable prediction of field behavior, although for some cases in overconsolidated clay slopes it was necessary to assume very high pore pressures or to reduce the effective cohesion to zero in order to explain slope failures. Henkel (1955) suggested for long-term design of slopes in London Clay that $c' = 0$ and ϕ' is as obtained from the laboratory tests. A mechanism important for the long-term stability of slopes was pointed out in 1953 by Bishop and Henkel: They showed that unloading due to excavation initially will not be reflected in the effective stresses but only in the decrease of the pore pressures. With time these negative excess pore pressures dissipate until eventually they are everywhere in equi-

librium with the steady seepage flow pattern appropriate to the new slope profile. This process which is correlated to a decrease of the average principal effective stresses can lead to slope failure. In the early 1960's more and more case histories were reported in overconsolidated clays and clayshales which could not be explained even with the previous modifications to the effective stress analysis. In 1960 Gould presented a number of cases of slides in highly overconsolidated clays from California. On the basis of stability analysis on slopes in these materials he suggested:

> "that the strength actually mobilized in slides or mass creep is inversely proportional to the amount of displacement which has occurred previously in the shearzone."

He also proposed

> "that previous slides or even small movements have permanently damaged the shear resistance of the stiff clays."

The deformations in a slope had been observed as displacements of a rigid mass on a narrow shear zone. He proposed therefore:

> "that a relatively small movement may suffice to reduce the strength available to the laboratory 'ultimate' value following brittle failure."

In 1964 Skempton proved the correlation between laboratory residual strength and the strength on natural slip surfaces on the basis of the detailed study of the failure at Walton's Wood. The phenomenon of residual strength for undisturbed clays had already been found in 1937 by Tiedemann and Hvorslev, and the term "residual strength" had been introduced by Haefeli in 1938. Skempton, however, could

correlate the concept of residual strength to the strength along natural shear planes on the basis of clear field evidence. He suggested that some mechanism of progressive failure might have caused the low strength values. He defined progressive failure in the following manner:

> "If for any reason a clay is forced to pass the peak at some particular point within its mass, the strength at that point will decrease. This action will throw additional stress on to the clay at some other point, causing the peak to be passed at that point also. In this way a progressive failure can be initiated and in the limit, the strength along the entire length of a slip surface will fall to the residual value."

He pointed out that in general

> "a slip may occur before the residual strength is attained throughout the clay, but once progressive failure has started the average strength of the clay will decrease inexorably towards the limiting residual value."

In the following years it became very common to analyse instability in terms of residual strength. In 1965 Skempton pointed out the importance of testing representative samples in a representative manner. He summarized the conditions by which discrepancies between laboratory and field data can arise, such as drainage conditions, rate effects, scale effects, testing the right material, and mechanisms of progressive failure. In 1966 Bjerrum suggested a possible mechanism for progressive failure, postulating that this mechanism was the result of the large content of "recoverable strain energy" in overconsolidated plastic clays. The conditions for this mechanism can be summarized as follows:

1) Unstable stress strain curve

2) Local stress concentrations on the foot of the slope

3) Large movements due to release of locked-in strain energy

Bishop (1967) proposed a mechanism based mainly on local overstressing. He explained that

"--- due to local overstress a zone of plastic equilibrium is formed in a slope long before general failure takes place."

He also pointed out that a change in loading conditions or pore pressures will lead to a

"--- progressive extension of the zone of failure along the potential slip surface, while within this zone the shearing resistance will commence to drop from its peak to its ultimate or residual state."

It was suggested that the drop in strength depends on the difference between peak and residual strength, which he described by the

"Brittleness Index": $I_B = \frac{s_f - s_r}{s_f}$

where s_f and s_r are respectively the peak and residual strengths. Bishop defined progressive failure the same way that Terzaghi did in 1948:

"Progressive failure indicates the spreading of failure over the potential surface of sliding from a point or line towards the boundaries of the surface. While the stresses in the clay near the periphery of this surface approach the peak value, the shearing resistance of the clay at the area where the failure started is already approaching the much smaller ultimate value. As a consequence the total shearing force that acts on the surface of sliding at the instant of complete failure is considerably smaller than the shearing resistance computed on the basis of the peak values."

Peck (1967) discussed a set of Conlon's test results and his interpretation of them with respect to progressive failure on overconsolidated clays. The strain at which the peak stress occurred was found to depend

on the normal pressure. In relating this result to a circular failure surface in a slope, Conlon suggested that since the stress distribution is nonhomogeneous, the strength has to be mobilized in a nonhomogeneous manner. It was concluded that the maximum resistance cannot be as great as the resistance that would be calculated on the basis of the peak strengths nor as small as the resistance based on the residual strengths. Conlon had suggested further that fissures in a soil could lead to high compressibility and permit differential strains to occur along the surface of sliding. Peck pointed out, too, that

> "--- possibly the most important factor affecting our ability to predict whether or not a slide will start, is whether we are in an old slide area."

The importance of old shear zones for the overall stability of slopes was recognized for the design of open pit mines in many parts of the world (e.g. Schultze, 1956; Dolezalek and Duero, 1962; Popov, 1948 and 1960; Komarnitskii, 1968). A number of case histories have been reported in which slope stability was clearly defined by old shear zones (e.g. Skempton, 1966; Esu, 1967; Leussink and Mueller-Kirchenbauer, 1967; De Beer, 1969). Turnbull and Hvorslev in 1967 discussed the problem of progressive failure referring to nonhomogeneous stress distribution and local overstressing as described by Bjerrum and Bishop. It was pointed out that possible limitations of these mechanisms were indicated by several case histories in overconsolidated soils in which the average shear strength along failure surfaces was found

> "close to peak strength, although a smaller average shear

strength would be expected for soils having a residual strength which is appreciably smaller than the peak strength."

In 1969 Duncan and Dunlop studied the effects of initial lateral stresses on the stresses around a slope. They analysed a homogeneous linear elastic isotropic material using a plane strain formulation of the Finite Element method. The stress conditions in a slope after excavation were found to be strongly influenced by K_0, while a variation of Modulus of Elasticity within the ground had very little influence. It was concluded that for high K_0-values large shear stresses, even failure stresses, might develop at some points within a slope even though the Factor of Safety (in terms of undrained strength $\phi_{cu} = 0$) was far greater than one. They proposed that the existence of high horizontal stresses in heavily overconsolidated clays and shales increases the probability of progressive failure. Yudhbir (1969) showed in oedometer tests that K_0-unloading causes shear failure of the specimen. He concluded that in areas where unloading had occurred, the ground fails due to K_0-unloading down to a depth of about 40 feet. Additional shearing strains associated with further unloading to small stresses were believed to reduce the shearing resistance of the material along the shear surface. Christian and Whitman (1969) studied the effect of horizontal stress release on progressive failure using a theoretical model. They considered a single layer bonded to a rigid base with elastic, plastic, and strain softening behavior for the bonding between the layer and the base. In this model it was found that even for slopes with a high Factor of Safety

in terms of residual strength, there is a possibility of propagation of a failure surface.

Bishop and Lovenbury (1969) studied the long-term creep characteristics of overconsolidated London Clay and normally consolidated Pisa Clay under drained conditions. They showed that long-term loading does not lead to substantial strength reductions. This suggests that there is no path to the residual which by-passes the peak. James (1970 and 1972) presented an investigation of about 90 case histories most of them in overconsolidated English clays. The majority of slides had failed with $c' = 0$ and $\phi'_s = \phi'_{peak}$. Many of the failures which had occurred at reduced c' and ϕ' were found to be reactivated slides along old failure planes. The number of slips exhibiting progressive failure was reported to be very small. In this context James defined progressive failure

> "as the simultaneous or quasisimultaneous decay in both parameters, c' and ϕ'."

Investigations of repeated slides showed that a reduction in strength due to failure happens only after very large movements of the order of several feet. It was suggested that unless deformations are localized, along one thin layer or at the interface between two somewhat different layers, the strains would be too small to give anything approaching the residual. Progressive failure might therefore not be possible in a homogeneous slope. Wilson (1970) reported a large number of landslides which had failed in part by showing progressive

movements, and referred to them as examples of progressive failure.

In the light of James' observations Skempton (1970) again discussed first-time slides in overconsolidated clays and described the post peak changes in strength as comprising two successive stages:

"a) Dilatancy and the opening of fissures leading to increases in water content and culminating in a drop in strength to the fully softened value, at which stage there is a softened shearzone with numerous discontinuous shears.

b) Development of principal shears of appreciable length, some of which eventually link together and form a continuous shear, when the residual strength is reached along the entire slip surface."

Referring to James' data he explained that there is sufficient field evidence to show that

"the strength in first-time slides in London Clay cuttings tends towards, and does not fall significantly below, the fully softened value."

The end of the softening stage (a) was believed

"to represent a more realistic limit than the residual for the ultimate drop in strength preceeding a first-time slide in London Clay."

This statement is a reiteration of the opinion expressed on several occasions before 1964 (e.g. Henkel, 1955; Skempton, 1948). Nevertheless, he did not preclude the possibility that strength reduction prior to a first-time failure is more substantial in some materials, as proposed by Bjerrum (1966). Bishop (1971), on the other hand, has argued that a nonuniform mobilization of strength from peak to residual along the slip surface might also account for known cases of first-time failures which had been cited as failures at $c' = 0$ and

$\phi'_s = \phi'_{peak}$. Bishop pointed out

"that this hypothesis applies to clay slopes whether or not the clay is fissured."

In 1971 Bishop et al. described the investigations of the post peak behavior of clays in a new ring shear apparatus. From the results obtained they concluded that residual strength conditions in clay can be reached only after very large strain. The initial loss in strength was explained by a destruction of cohesion due to dilatancy and break of cementation bonds.

Progressive failure has also been investigated and described in other materials. For granular media very detailed studies exist, e.g. by Taylor (1948), Wroth (1958), Roscoe, Schofield and Wroth (1958), Rowe (1962 and 1969), Roscoe (1964 and 1965), Muhs (1965). Quick-clays were investigated e.g. by Bjerrum (1955), Bjerrum and Landva (1966), Conlon (1966), Bjerrum et al. (1969), Townsend et al. (1969). Progressive failure in rocks was described e.g. by Terzaghi (1962), Mueller (1962), Haefeli (1965), Krsmanovic (1967), and progressive failure in snow slabs was reported about by Haefeli (1965).

2. Outline of the Project

As can be seen from the previous review, the problem of progressive failure in overconsolidated clays and mudrocks is by no means resolved. There are many different concepts and definitions in use, which, because they were not clearly stated, have caused misunderstandings and misinterpretations of slope failures in overconsolidated

clays and mudstones. There is an apparent tendency to restrict pre-failure strength reductions to a softening mechanism as Terzaghi first suggested (Skempton, 1970), but it is still not yet fully accepted that a reduction in strength to residual is unlikely in a slope before first-time failure (Bishop, 1971). The restriction of pre-failure strength reductions to softening mechanisms is mainly based on evidence in London Clay. Hence it appears worthwhile to have a closer look at first-time slides, which have been reported in other parts of the world. The materials involved, however, embracing overconsolidated clays as well as shales and mudstones, are not yet consistently classified (Underwood, 1966). In order to come closer to a solution of the problem of progressive failure the following points will be considered in this study:

1) The term progressive failure needs to be clearly defined. The implications, limitations, and consequences of the definition should be discussed and compared with previous investigations.
2) The study on first-time slides by James (1970) should be extended to other materials, to find out whether the results in English clays hold true for materials in other parts of the world.
3) A classification system should be established to permit qualitative and quantitative comparisons of different materials on the basis of some of their most significant engineering

properties. A systematic study of some of the most conspicuous properties, such as "slaking" or "softening" is of interest to add to the understanding of the process which leads to strength reduction in overconsolidated clays and mudstones.

4) The mechanisms which can cause apparent strength reductions in a slope should be analysed in order to distinguish them from the mechanisms leading to true strength reduction.

CHAPTER II

DISCUSSION OF PREVIOUS STUDIES

1. A Definition of Progressive Failure

Different concepts and definitions of progressive failure are in use. Therefore a systematic investigation and a thorough discussion of the problem is warranted to clarify not only the concepts but also the terminology. For example, Hayley (1968) and Wilson (1970) described slope failures showing progressive movements which were generated in various manners. They referred to them as examples of progressive failure. Wilson summarized that the "progressive nature of landslides", as indicated by "progressive movements of the sliding soil mass", was found "to be in agreement with generally accepted hypotheses concerning the mechanism of such progressive failures."

Progressive movements within a slope certainly can be indicative of progressive failure since progressive failure is always correlated to some kind of progressive movements. However, progressive movements are not necessarily related to progressive failure. Progressive movements can also indicate reactivated failures.

In the present study progressive failure will be defined in the following manner:

> "Progressive Failure exists in the field, when on average the strength mobilized at failure is less than the peak strength exhibited by testing a representative element of the slope in a representative manner."

In order to understand the implications of this definition some of the terms used will be discussed in the following.

1.1. Representative Elements of the Slope

The term "representative element of the slope" implies that the samples to be tested should be such that they represent those elements of a slope which define its overall strength. These samples have to be representative in structure, and therefore have to be of a representative size. A representative sample must be sufficiently large to contain a representative selection of all particles and all discontinuities in the clay. Sufficient sample size is particularly important for overconsolidated clays and mudstones which generally contain joints and fissures. The representative size of the sample depends on the fissure spacing. The influence of sample size on strength has been studied systematically in terms of undrained strength parameters, e.g. for London Clay (Agarwal, 1967; Skempton et al., 1969). It was found that the in situ undrained strength of London Clay is around 65 to 75% of the conventional 1 1/2"x3" triaxial compression strength. Less systematic investigations exist for the effect of sample size on the effective strength parameters c' and ø' of stiff fissured clay. From investigations for Barton Clay (Marsland and Butler, 1967) it was found that for the stress levels typically encountered in landslide problems (500 psf to 1500 psf) there is little

difference between the strengths obtained from large in situ shear box tests and those obtained from 3 inches to 5 inches triaxial specimens. The strength values of small intact specimens (1 1/2" diameter) are however substantially higher, which is mainly reflected in the cohesion values. Results similar to the ones obtained for Barton Clay were obtained for London Clay (Bishop, 1967).

A representative structure is also influenced by disturbances during sampling. Disturbances of the original structure during sampling are most significant in soft clays leading to substantial reduction of undrained strength values (Skempton and Sowa, 1963; Ladd and Lambe, 1964; Noorany and Seed, 1965). Effective strength parameters are very likely influenced much less by sampling (Skempton et al., 1969). Stiff intact clays are probably least affected by sampling disturbances. For stiff fissured clays the undrained strength might be reduced during sampling (Ward, Marsland and Samuels, 1965), but if the clay is not too hard and brittle there might be little influence (Skempton and Larochelle, 1965). The effective strength parameters are probably not sensitive to minor sampling disturbance (Skempton et al., 1969).

A very important point in the assessment of the strength of a slope is the proper selection of samples from the zones of the slope which are relevant for its stability. Natural materials almost always show zones of nonhomogeneity either in terms of material or in terms of structure. If these zones are zones of weakness they ob-

viously influence the stability of the slope. Since they might represent potential failure zones, the strength along these zones should be tested.

Structural discontinuities have been classified by Skempton and Petley (1967) into:

a) Continuous shear planes: they are caused by large shearing displacements. $c' = 0$ and $\phi' = \phi'_{residual}$ are the governing strength parameters.

b) Discontinuous shear planes (slickensides): they are caused by small displacements. $c' = 0$ and $\phi' = \phi'_{peak}$ are the governing strength parameters within such a zone.

c) Joint surfaces: they are discontinuities along which no movements have occurred. $c' = 0$ and $\phi' = \phi'_{peak}$ are the governing strength parameters.

The characteristic strength parameters were obtained from shear tests along fissures and joint surfaces on several overconsolidated clays from England and on Siwalik Clay from the Mangla Dam project (Pakistan).

Zones of weaker materials can be

a) soft clay layers, or

b) subsequently filled discontinuities.

Structural discontinuities frequently exist within zones of weaker materials, particularly along the interface of soft and hard layers (e.g. Skempton, 1965 and 1966).

The importance of discontinuities on the overall stability

of slopes has long been recognized throughout the world. Their significance has been noted for the slopes of large brown coal open pit mines in Germany (e.g. Schultze, 1956; Dolezalek and Duero, 1962) as well as for the slopes in open pit mines in the USSR (e.g. Popov, 1948 and 1960; Komarnitskii, 1968). Peck (1967) pointed out the importance of old shear zones when he suggested that it might often be more relevant to ascertain whether we are in a slide area than to apply a sophisticated slope analysis. The importance of discontinuities on slope stability is illustrated by many studies (e.g. Esu, 1967; Leussink and Mueller-Kirchenbauer, 1967; De Beer, 1969; Wolters, 1969; Duero, 1970; Kloiber, 1970). Some results from strength tests along zones of weakness in clays and mudstones are summarized in Table II.1.

There are many reasons for the presence of zones of weakness. The zones of weaker materials can be either sedimentological features or caused by the subsequent filling of discontinuities with softer materials such as clays. Structural discontinuities can occur during shear movements which cause continuous and discontinuous shears, or as a result of tension which causes joint surfaces. Zones of weakness are often difficult to detect by the usual sampling methods. It is generally easier to recognize the factors which cause discontinuities than the discontinuities themselves. Therefore the mechanisms causing discontinuities will be discussed in the following.

Landslides

A great number of natural slopes in clays and clay shales have been affected by some kind of landsliding (e.g. Peck, 1967; Leussink and Mueller-Kirchenbauer, 1967; De Beer, 1969). Most of these slides were combined with large displacements which resulted in continuous shear zones. In addition to shear zones a series of tension cracks must be expected in failed slopes.

Solifluction

Continuous shear planes as well as discontinuous shears have been observed beneath post glacial solifluction sheets in South West England down to a depth of 16 feet. Solifluction sheets can be characterized by prominent lobal features (e.g. Skempton and Petley, 1967) but very often may not be indicated by any topographic expression (Weeks, 1969).

Tectonic movements

Tectonic movements (Skempton, 1965 and 1966; Esu, 1967; Wolters, 1970; Duero, 1970; Kloiber, 1970; Skempton and Petley, 1967; Hammon and Post, 1969) can cause continuous shear surfaces or shear zones which contain numerous slickensides as well as tension cracks. The strength reduction by interlayer slip during folding is particularly high along the interface of harder and softer materials, e.g. between soft mudstone and harder sandstone (Skempton, 1966).

Bedding plane slip associated with anticlinal rebound in response to valley formation (Matheson, 1972)

Anticlinal rebound in response to valley formation can be

observed in river valleys cut into the generally flat lying strata of the North American prairie regions. Slip planes, probably due to bedding plane slip as a consequence of the anticlinal rebound, are noticible in several locations along the interface of softer and harder layers. However, strength tests along the slip planes were not performed.

Differential compression of sediments

Minor shear zones and slickensides are characteristic features in coal measure rocks and underclays all over the world (Schultz, 1958; Teichmueller, 1968; Best et al., 1970; Kloiber, 1970). They are probably generated by differential movements during the compression of these layers and the underlying materials.

Glacial ice movements

The effect of glacier movements on the underlying soil has been noted by Graftio (1936) for overconsolidated clays in Russia. He found continuous shear zones in soft clay zones which lay between harder strata, and proposed that these soils had undergone a huge shear test during the movements of the glacial ice masses. In the prairie regions of North America contorted bedrock is a common occurence which has been traced to glacial ice movements (Bayrock and Hughes, 1962). Along the contorted beds, shear zones due to slip must be expected, particularly in bentonitic beds. Bentonitic clay contains a substantial unfrozen water content at temperatures normally encountered in permafrost. Therefore, in regions of permafrost, bentonite beds represent

particularly weak zones when compared with the neighbouring solid frozen materials. They are most prone to failure under applied shear stresses. Backofen (1957) described slickensides in overconsolidated clays in northern Germany and referred them to ice movements as well as to differential movements between clay clusters during thawing of permafrost.

Differential swelling of highly swelling clays constrained between non-swelling materials

Differential swelling along the interface of non-swelling and swelling materials may provide shear strain large enough to reduce the strength of the swelling zone along the bedding planes possibly even to residual strength. This process must be considered very slow, so that it might be impossible to observe it in the field.

Sedimentological features

Along the bedding planes of some clays, strength values have been measured which are lower than those in the adjoining clay mass, even though no prior deformations were indicated (Dolezalek and Duero, 1962; Komarnitskii, 1968; Esu and Calabresi, 1969). The reduced strength was mainly reflected in reduced cohesion values.

1.2. Testing in a Representative Manner

Strength test results can be greatly affected by the testing procedure, which should be done in a representative manner.

Differences in the orientation of the samples may result in considerable differences between the shear strength along a slip plane in the ground and the laboratory strength (e.g. Hvorslev, 1960; Uff and Nash, 1967; Esu and Calabresi, 1969; Conlon, Tanner and Coldwell, 1971).

Strength test results can also be influenced by the rate of testing. Undrained peak strength values are particularly sensitive to rate of strain (Casagrande and Wilson, 1951; Bjerrum, Simons and Torblaa, 1958; Hvorslev, 1960; Richardson and Whitman, 1963; Skempton and Larochelle, 1965). The observed drop in strength with time to failure varies from material to material. Therefore, for a dependable stability analysis, in terms of total stresses, the strength tests should be performed at the same rate at which the slope failure occurs (which is by no means easy), unless appropiate correction factors are determined and applied.

Little information is available on the effect of strain rate on the effective strength parameters of clays. The influence of rate of strain on peak effective strength was reported by Bishop and Henkel (1957) on remoulded Weald Clay from England, as well as by Bjerrum, Simons and Torblaa (1958) on a normally consolidated marine clay from Oslo, by Hvorslev (1960) on highly overconsolidated Wiener Clay, and by Morin (1972) on normally consolidated Champlain Sea Clays. Most of these investigations showed an influence of strain rate on ef-

fective peak strength, however it was far less pronounced than for undrained strength. From consolidated drained tests on normally consolidated Oslo Clay no influence of strain rate on effective peak strength could be detected (Bjerrum, Simons and Torblaa, 1958). Hvorslev (1960), referring to some test results on highly overconsolidated Wiener Clay, suggested that the influence of strain rate on effective peak strength was more significant for overconsolidated materials than for normally consolidated clays. Results from drained constant stress creep tests on overconsolidated London Clay and normally consolidated Pisa Clay supported this theory (Bishop and Lovenbury, 1969).

Very little variation of effective residual strength was found in studies on Edale Clay from England (Skempton et al., 1969), and can be considered neglegible even for very slow rates of movement.

2. Conditions for Progressive Failure

Progressive failure is possible only in materials which show an unstable stress-strain curve, which is indicated by a drop in strength from peak to residual at large strains (e.g. Haefeli, 1967; Peck, 1967; Bjerrum, 1967; Bishop, 1967). Bishop (1967) introduced the

"Brittleness Index" $$I_B = \frac{s_f - s_r}{s_f}$$

where s_f and s_r are respectively the peak and residual strengths. He suggested that the Brittleness Index might indicate the tendency of

a soil to experience progressive failure.

The present study of progressive failure will be limited to overconsolidated clays and mudstones only and does not deal with other materials, such as quick clays or loose sands.

The problem of progressive failure will be considered only in terms of effective stresses.

Strength loss can be expressed for overconsolidated clays and mudstones in terms of

1) loss of cohesion c' only
2) loss of cohesion c' as well as reduction of angle of friction ø'.

Both kinds of strength reduction have been discussed in the literature (as shown in Chapter I) and will be treated further in the following section. In addition mechanisms of apparent strength reduction will be described, which if not recognized, can be misinterpreted as strength reductions within a slope.

2.1. Strength Reduction affecting Cohesion c' only (Softening)

Softening is the mechanism which was first suggested to explain time-dependent strength decrease as it had been observed in slopes of overconsolidated fissured clays (Terzaghi, 1936). This mechanism has been somewhat neglected since 1964 but its importance became clear again after the reinvestigation of a large number of

slope failures in English clays (James, 1970). The reduction of cohesion can be explained by several processes which will be discussed in the following.

Stress release causing opening of fissures (Terzaghi, 1936; Skempton, 1948)

Stress release causing opening of fissures and softening of the clay can start at the face of the open cracks. The softened material eventually deforms, a stress redistribution occurs and finally the clay becomes a normally consolidated or "fully softened clay" (Skempton, 1948). The destruction of the original soil structure results in a complete loss of the cohesion intercept. This softening process is not dependent on large deformations within the slope before failure but is dependent on the presence of fissures and joints within the soil. The rate of softening is influenced by the type of clay and the climatic conditions. It is likely to be rather slow in general and noticible strength decrease can be realized only after decades.

Softening due to displacements (Skempton, 1970)

Skempton (1970) pointed out that in a soil subjected to shear movements two stages of strength reduction can be distinguished, a "fully softened condition" and the "residual condition". He explained that

> "the displacement required to reduce an overconsolidated clay to the fully softened condition, or approximately to this condition, is several times greater than the displacement at peak strength, but is nevertheless not large, and is considerably less

than that corresponding to residual strength. Moreover it seems probable that when the strength has fallen to the fully softened value, or close to it, there is yet no principal shear surface but instead a complex of minor shears such as the Riedel thrust and displacement shears (Skempton, 1966) which have not been linked into a smooth continuous surface. Particle re-orientation will have occurred along these minor shears, as demonstrated by Morgenstern and Tschalenko (1967), but the effect of such a fabric is already built into the fully softened strength."

This fully softened state of strength agrees well with the strength at the "critical state" in terms of critical state soil mechanics (Schofield and Wroth, 1968; Roscoe, 1967).

Skempton's softening process which is caused by dilatancy during straining and particle orientation along minor shear is initially quite different from the initial stages of Terzaghi-softening, which is mainly due to a destruction of the original clay structure by swelling. Prerequisite for Skempton-softening are displacements large enough to reach this stage of fully softened strength; these displacements are certainly larger than the deformations needed to open the fissures which represent the starting condition for the Terzaghi-mechanism. Skempton-softening is not dependent on the presence of fissures or joints; however, it might be reached sooner in zones with discontinuities than in intact materials (Skempton and Petley, 1967). It can be imagined that eventually Terzaghi-softening becomes similar to Skempton-softening, when differential displacements occur.

It should be noted that although residual strength values

might be reached locally along the minor shears the overall strength along the shear zone is equal to the fully softened strength. As long as there is no continuous shear plane which develops only after very large displacements (James, 1970), the strength has not reached its residual value.

Softening due to weathering

The different processes of weathering such as slaking, freezing-thawing, oxidation etc. can cause complete destruction of the clay structure and a consequent loss of strength. Softening due to weathering is generally restricted to relatively shallow depths below the ground surface. The macrostructure plays an important role in determining the depth reached by weathering action (Esu, 1967). The thickness of the cover of softened clay was found to be greater in zones where the clay was more intensely jointed and fissured, e.g. in fault zones and where the slopes had been affected by a series of landslides in the past. The slides in weathered clays were described by Esu (1967) as

> "flows in which no definite surface of sliding seemed to exist and which occurred on very gentle slopes."

The lower boundary of the sliding mass often coincided with the base of the cover of weathered clay (Esu, 1967; James, 1970; Einsele, 1961; Ringheim, 1964).

Weathering in temperate and cold climates can be explained mainly by physical processes, which do not change the mineralogy of

the soil and to a lesser extent by chemical processes. This is indicated by the fact that no marked variations in mineralogy have been noted in Italian clays (Esu, 1967) and most English clays (James, 1970; Spears et al., 1970). The Atterberg Limits show approximately the same values for weathered and unweathered clays. However, Chandler (1967) described different Atterberg Limits for unweathered and weathered Triassic silty mudstone from England. Moreover, a very substantial decrease of carbonate content due to weathering (from 20% to 1.7%) has been observed.

Chemical weathering results mainly in the solution of the original cementing agents such as carbonates, or in the change of the chemistry of the pore fluid of the soil. Chemical weathering probably can reach larger depths than physical weathering. Effects of chemical weathering can be observed in temperate climates to depths of more than 20 feet.

The main physical weathering processes are slaking and frostaction. Slaking is defined as the disintegration of argillaceous materials upon cyclic drying and wetting. These processes support each other, the one process creating joints and fissures which accelerates the other one. Rate and amount of disintegration due to slaking depend on the amount of swelling clay minerals and the environmental conditions. Slaking is a very severe process which causes a progressive destruction of the soil structure. The final

result may be a remoulded clay or silt. Under extreme conditions the final water content can reach the Liquid Limit of the soil.

Field evidence of the softening processes

Reduction of strength due to softening can be confirmed by field observations of first-time slides which failed at reduced cohesion and peak friction angle. James' investigation (1970) of about 90 cases of slope failures in English overconsolidated clays showed that the majority of first-time failures were failures at $c'=0$ and $\phi'=\phi'_{peak}$. This study is consistent with strength decrease due to softening and suggests that softening is the kind of strength reduction which is most common in nature. Other well-known examples are the slides along the Panama Canal which were believed to be first-time slides at residual strength parameters (Bjerrum, 1967). Recent detailed investigations indicated that failure took place at $c'=0$ and $\phi'=\phi'_{peak}$ (USCE, 1970). The decrease of the strength with degree of slaking was measured for a mudstone in the field by Hammon and Post (1969). They found that the final shear strength of the slaked mudstone had reached a value which was equal to the strength of the remoulded material. Leussink and Mueller-Kirchenbauer (1967) described a zone of weakness which had developed due to weathering. An originally compact shale changed to silty clay after the shale was brought close to the ground surface. Strength tests showed that the effective cohesion had dropped to zero. In the investigation of the Devon Slide (Eigenbrod and Morgenstern, 1971) mudstone was

found in the back slope of the slide. Effective strength tests showed a far lower cohesion value c' for partially weathered rock than for the unweathered rock.

2.2. Strength Reduction affecting Cohesion c' and Angle of Friction ø'

A reduction in strength from peak to residual was noted in the field first by Gould (1960) and Skempton (1964). Gould explained the strength reduction by displacements associated with previous slides or movements. Skempton suggested that some form of progressive failure must be operative to take the clay past the peak. He thought "that once progressive failure has started the average strength of the clay will decrease inexorably towards the limiting residual value", unless the slope has not failed before. Several of the many mechanisms which had been suggested to initiate this process will be discussed in the following.

Local overstressing

The mechanisms of local overstressing can be divided into

a) stress concentrations

b) nonhomogeneous stress distribution along the failure plane.

Skempton (1964) suggested that cracks, holes and other imperfections in the soil act as stress concentrators which could force a clay to pass the peak at some particular point within its mass and thus initiate progressive failure. A similar mechanism was suggested by

Bishop in 1967. Duncan and Dunlop (1969) showed in a Finite Element stress analysis that local stress concentrations can develop in a slope where large horizontal stresses had been removed. Taylor (1948) pointed out the importance of nonuniform stress and strain correlations for the development of progressive failure in sands under both laboratory and field conditions. According to a nonuniform stress distribution along a potential slip surface, a nonuniform strength distribution results, which at some point within the soil mass might carry the soil past peak strength. Bishop (1967) discussed the effect of nonuniform stress distribution in more detail. He showed that

> "as the effective stresses change in a slope (due to excavation, construction of foundations, erosion, or rise in pore pressure under constant loading) the shear stresses along a potential failure surface will rise in a nonuniform manner. If locally the shear stress or the ratio of shear stress to effective normal stress reaches the limiting value, local failure will result".

This differs from Skempton (1964) who pointed out that once progressive failure has started it will tend to reach residual conditions everywhere along the potential slip plane. Bishop explained that only further change in the load conditions will lead to a progressive extension of the zone of failure along the potential slip surface, while within this zone, the shearing resistance will begin to drop from its peak to its ultimate or residual state. He believed that small further displacements which will be associated with a decrease in overall shearing resistance are sufficient to

bring

"the whole surface into the peak and post peak state. Complete failure will then have occurred and the strength at all points will lie between the two limits of the peak and residual states."

Conlon (Peck, 1967) also proposed that the peak strength cannot be mobilized simultaneously along the surface of sliding. He pointed out that the influence of a nonhomogeneous stress distribution might be augmented since brittleness decreases with stress level. At low stress levels peak shear strength will be reached after smaller displacements than at higher stresses. Conlon further proposed that due to fissures in a clay and consequent variable compressibility differential strains may occur along the surface of sliding, thus increasing the chance of progressive failure. Bishop (1970), however, pointed out that nonhomogeneous stress distribution and hence progressive failure is not dependent on joints or fissures.

Leussink and Mueller-Kirchenbauer (1967) described a nonuniform shear stress distribution along a predetermined sliding plane which consisted of a soft clay zone of varying thickness. It was suggested that in the thicker regions of the clay layer, a given displacement along the sliding plane caused smaller differential shear movements and hence mobilized lower shear stresses than those in the thinner parts of the clay zone.

Nonhomogeneous stress distribution and hence nonhomogeneous mobilization of shearing resistance has been described by

Krsmanovic (1967) for jointed rock masses. He pointed out that the distribution of normal stresses along the potential failure plane depends on the magnitude of the shear deformations and is constantly changing.

Stress release

It is well known that a release of horizontal stresses can lead to failure within a soil mass (Terzaghi, 1936; Skempton, 1948). Duncan and Dunlop (1969) showed by using the finite element technique that excavations in a highly overconsolidated soil with high K_0 values and consequent release of horizontal stresses causes the soil to reach failure locally. This nonhomogeneous stress distribution was believed to initiate progressive failure. Yudhbir (1969) suggested that K_0 unloading, resulting from unloading of vertical stresses, would also lead to failure conditions within the ground and carry the soil past its peak. Both mechanisms imply that very little deformation would lead to strength reduction. Christian and Whitman (1969) tried to illustrate the effect of horizontal stress release by developing a theoretical model for the progressive failure of a single layer bonded to a rigid base. Elastic, plastic, and strain softening behavior was considered for the bonding between the layer and the base. They found that even for slopes with high factors of safety in terms of residual strength, there is likely to be extensive propagation of a failure surface, which depends to a high degree on the depth of the excavation. This result would suggest that the development of

a failure surface does not necessarily cause failure of the overall slope. The analysis however is based on a simplified stress-strain curve which shows an abrupt drop in strength from peak to residual. Yielding and residual strength values in this case can be reached after displacements which are only slightly larger than displacements for peak strength. This behavior is certainly not true for real soils (James, 1970; Skempton, 1970). The analysis further implies that failure takes place just along the interface between layer and base. This assumption of an infinitely narrow failure zone is definitely not valid for a natural material. The development of a zone of failure in clays and the resulting stress redistribution is much more complex (Skempton, 1966; Morgenstern and Tchalenko, 1967).

Bjerrum's mechanism (1966)

Bjerrum's mechanism is based on three main conditions:

1) unstable stress-strain curve
2) local stress concentrations on the foot of a slope
3) large movements due to release of locked-in strain energy.

The importance of large movements for carrying a soil beyond peak was well recognized. In order to induce large movements Bjerrum introduced the concept of "release of locked-in strain energy". It was suggested that the release of stored energy occurs during the destruction of diagenetic bonds due to weathering. This causes large

volume changes and hence large movements adjacent to restraints. It appears however, that the large volume changes due to weathering are mainly the result of outside energy rather than the release of "stored energy". Processes such as slaking break down the clusters in which clay particles are bound together (Balasubramonian, 1972), thus increasing the effective surface area from a relatively low value for the clusters to the far higher value of the surface area of the actual minerals. An increase in surface area is equivalent to an increase in swelling potential.

Overstraining

James (1970) suggested that very large movements (in the order of several feet) are pre-requisite for strength reductions in a slope from peak to residual. His proposal was based on the investigation of more than 90 case histories and on Lovenbury's investigation (1969) of longterm creep in which was found that there is no way to by-pass the peak. Skempton (1970) proposed that the post peak behavior is composed of two stages:

a) the fully softened stage

b) the residual stage.

The residual stage is thought to be reached after very large strains when a continuous shear surface has developed. Consideration of two stages for post peak behavior agrees well with Wroth's tests (1958) on randomly packed steel balls in simple shear. He could differentiate between

a) the strength at the critical state

b) the final strength value after very large displacements, which correspond to a development of regularity of packing in the material.

Comparison with field evidence

The mechanisms, which were suggested to reduce c' and ϕ', have been well established on a hypothetical basis, but not by any single unambiguous field case. Bishop (1971), however, believes that a nonuniform mobilization of strength from peak to residual along the slip surface might also account for known cases of first-time failure. He showed that when assuming a nonuniform mobilization of shearing resistance as proposed by Conlon (Peck, 1967) slope failures could be explained, which also had been interpreted in terms of c' = 0 and $\phi'_{mobilized} = \phi'_{peak}$. His examples, nevertheless, do not appear unambiguous. On the other hand there are well documented cases of landslides in soils with unstable stress strain curves in which the average shear strength mobilized at failure was close to peak. If nonhomogeneous stress distribution had initiated progressive failure and consequently had caused total failure of the slope, then the strength along the slip plane should have been mobilized nonhomogeneously, indicated by an average shear strength at failure less than peak (Turnbull and Hvorslev, 1967). Other cases, which throw doubt on the importance in practice of the concepts of nonhomogeneous stress distribution in a slope, are the observations of high, steep slopes

in overconsolidated clays which had been found stable for many decades, even for centuries. Failures were observed only along zones of weakness. Stable slopes in Italian clays have been described by Esu (1967), in English clays by Burland (1970) for Oxford Clay and by Best et al. (1970) for Ball Clay. Many stable slopes may also be found in Cretaceous clays of Southern Alberta. Nonhomogeneous stress distribution certainly occurs in these slopes and in some cases has even caused local failures (Burland, 1970), but it has not lead to the overall failure of the slopes. It should be noted that the above observations were generally reported for slopes in low plastic clays (mainly Kaolinitic clays), where Bjerrum's mechanism would not seriously effect the overall strength of the clay. De Beer (1969) described slopes in canal cuts in Belgium which generally are stable, many years after construction, and which were designed on the basis of peak strength parameters. Slides occurred only in locations where previous sliding surfaces existed. Bjerrum quoted a number of case histories of landslides which purported to prove his concept, but a closer reinvestigation of these and other cases showed that the slides were either slides along a pre-existing failure plane or failed at fully softened strength parameters. The Seattle Freeway slides, the Sandnes Slide, and the Volga slides, as well as the slides near the South Saskatchewan Dam, the slides reported by Einsele (1961, 1964), and the slide at Jackfield have been clearly identified as slides along pre-existing failure planes.

The East and West Culebra slides have been recognized as failures at softened strength conditions (USCE, 1970), which possibly were delayed by slow pore pressure equalization.

2.3. Apparent Strength Reduction (Delayed Pore Pressure Equalization)

Delayed pore pressure equalization (Bishop and Henkel, 1953) is a common reason for delayed failures of cuts in clays and mudstones. This phenomenon has caused many false interpretations of delayed slope failures, which have been referred to some kind of progressive failure. Only recently, due to improvement in piezometric installations has this mechanism been observed in the field. For example, the recent reinvestigations of the slides along the Panama Canal revealed pore water pressures which are still below the canal water level, 70 years after excavation (USCE, 1970). De Beer (1968) has also observed negative excess pore pressures in cuts in Boom Clay (Belgium) and Wood (1971) in coastal slopes in English clays.

In a numerical analysis the time for equalization of negative excess pore pressures caused by slope excavations was calculated. For 100 feet deep excavations in clay shales steady seepage conditions were found to be reached after 50 to 500 years, depending on the permeability of the ground. It was very interesting to note that the time delay of slope failures, commonly observed in London Clay, agrees well with the analysed time for pore pressure equalization in comparable slopes.

TABLE II.1.

SUMMARY OF SOME RESULTS OF STRENGTH TESTS ALONG ZONES OF WEAKNESS IN CLAYS AND MUDSTONES

Zone of Weakness	Material	Test	Strength Parameter for Intact Material: ϕ' peak degrees	c' peak psf	ϕ' residual degrees	Summarized Results	Additional Remarks	Author
Comparison of strength parallel (∥) and vertical (⊥) to bedding planes	Slightly over-consolidated stratified lacustrine clay from Welland (Canada)	Triaxial tests	a) $\phi'_{\parallel}=19$ $\phi'_{\perp}=25$ b) $\phi'_{\parallel}=17$ $\phi'_{\perp}=23$	$C'_{\parallel}=0$ $C'_{\perp}=200$ $C'_{\parallel}=0$ $C'_{\perp}=200$	15° 12°	∥ to bedding plane: $c'=0$ $\phi'_{residual} < \phi' < \phi'_{peak}$		Conlon, Tanner, and Coldwell, 1971
Comparison of strength parallel (∥) and vertical (⊥) to bedding planes	Overconsolidated clay (Italy)	Direct Shear test	$\phi'_{\parallel}=14$ $\phi_{\perp}=16$	310	11°	$\tau_{peak\ \parallel} < \tau_{peak\ \perp}$	Less deformations were needed to reach residual strength parallel to bedding plane than vertical to it	Esu and Calabresi, 1969
Bedding plane between two different clays	Calovian Clay (U.S.S.R.)	Large-scale in situ Shear test	15 to 21	300 to 540	--	$c' \simeq \frac{1}{4.0} \times c'_{intact}$ $\phi' = \phi'_{intact}$		Komarnitskii, 1968
Bedding plane between a) two clays	Underclays of Upper Miocene (Germany)	Direct Shear Test	22°	$c'>0$	9°	a) $c' \cong 0$; $\phi' \cong \phi'_{peak}$ b) $c' \cong 0$ $\phi'_{residual} < \phi' = 15° < \phi'_{peak}$		Schultze, 1956 and Dolezalek and Duero, 1962
a) Continuous Shear Plane b) Slaked Material	Thin Clay layer Shale (Java)	Triaxial tests For situ Shear test and Triaxial test	18° 25°		14°	a) $c'=0$ $\phi'=\phi'_{residual}$ b) $\tau_{slaked} \simeq \tau_{remoulded}$		Hammon and Post, 1969
Discontinuous Shear Zone	Barton Clay (U.K.)	a) Triaxial tests on different sized specimens b) Direct Shear test	27°	430	15°	a) $\tau_{large\ specimen} \simeq 0.8\tau_{intact}$ b) $\tau_{large\ specimen} \cong 0.5\tau_{intact}$	In bulk fissured clay behaved more like a shattered rock than a clay.	Marsland and Butler, 1967
a) Continuous Shear Plane b) Discontinuous Shear Zone c) Joint Surface	Overconsolidated Clays (U.K.) Upper Siwalik Clay (Pakistan)	Direct Shear test	12° to 14° 23	60 to 750 1200	11to16 12to14	a) $c'=0$ $\phi' \simeq \phi'_{residual}$ b) $c'=0$ $\phi' \simeq \phi'_{peak}$ c) $c'=0$ $\phi' \simeq \phi'_{peak}$	Along discontinuities residual strength could be reached after less deformations than for intact clay.	Skempton and Petley, 1967

CHAPTER III

ENGINEERING CLASSIFICATION OF INORGANIC, NONCALCAREOUS, SEDIMENTARY MATERIALS

1. Outline of the Problem

The fine grained inorganic, sedimentary materials are the predominant sedimentary materials in the earth's crust (Pettijohn, 1957). Because of their grain size and the complexity of their clay mineral fraction they are among the most difficult materials to completely analyse. Wide differences of opinion exist with regard to their classification and identification. Underwood (1967) stated:

> "The geology literature shows that the terminology used to describe the entire group of argillaceous sediments is not standardized, and, consequently, numerous inconsistencies have developed in the classification and nomenclature of shales and related materials."

Some authors designate the term shale to all argillaceous sediments including claystone, siltstone, mudstone, and marl (Krumbein, 1963; Ingram, 1953), whereas others designate the larger group as the mudstone or mudrock group and classify shale as a member of it (e.g. Twenhofel, 1937; Mueller, 1967). Twenhofel's classification system is defined in the following manner:

unindurated	indurated	after incipient metamorphism	metamorphic equivalents
	Mudstones		
Silt	Siltstone		Slate
+H_2O=Mud	+fissility=Shale	→ Argillite	→ Phyllite
Clay	Claystone		Schist

The boundaries between unindurated, indurated, and incipient metamorphic mudstones are of no primary interest to geologists and therefore have not been clearly defined.

For engineers, however, this distinction is important, because engineers tend to divide the materials that constitute the earth's crust into two categories, soil and rock.

"Soil is the natural aggregate of mineral grains that can be separated by gentle mechanical means. Rock, on the other hand, is a natural aggregate of minerals connected by strong and permanent cohesive forces. Since the terms 'strong' and 'permanent' are subject to different interpretations, the boundary between soil and rock is necessarily an arbitrary one." (Terzaghi and Peck, 1967)

For boundary materials, such as mudstones and related materials, this differentiation is particularly ambiguous. For example, a material like Bearpaw Shale from the Great Plains in North America, which is very similar to some varieties of Oxford Clay from South West England, is called in North America a compaction-shale or clay-shale, while in the U.K. it might be called a heavily overconsolidated clay (e.g. Morgenstern, 1967).

The importance of the degree of induration of fine grained

inorganic sediments has already been rated by Mead (1936), who separated shales (as he named mudstones and related materials) into two groups:

1) Compaction Shales, with little cementation

2) Cemented Shales, with significant amounts of cementing action.

Compaction shales slake and disintegrate when acted upon by water subsequent to partial or complete drying. Well-cemented shales do not disaggregate when placed in water after previous drying. He realized, however, that there is no sharp line of demarcation between these two types of materials. Mead suggested that slaking tests might throw much light on the nature of a shale.

The most complete presentation of the problem of classification and identification of mudstones from an engineering point of view has been given by Underwood (1967). He attempted to develop a classification for mudstones based on their engineering properties, but had to admit that a completely satisfactory engineering classification for shales could not be given because available data on engineering properties of mudstones were imprecise and testing methods were not standardized.

A satisfactory engineering classification and identification scheme is needed because of the many difficulties encountered when dealing with mudstones. Slope stability problems, heave and swelling and low durability to weathering are characteristic pro-

blems associated with these materials. Thus it is of interest to develop a system of classification that is based upon tests, which provide information on typical engineering properties and furthermore bear on the characteristic problems. This classification should be based on simple engineering properties which can be measured quantitatively within a reasonable period of time and hence are objective in nature.

1.1. Engineering Properties

Very detailed discussions of the significant engineering properties have been given elsewhere (e.g. Underwood, 1967), and thus only a short review of the most important properties will be presented.

The compressive strength of mudstones ranges from less than 25 psi for weaker compacted sediments to more than 15 000 psi for well--cemented mudstones. Unconfined compression tests are rather easy and fast to perform but the results are dependent on many factors, such as sample size, strain rate, and degree of drying and wetting. Effective strength parameters would be more characteristic properties for many mudstones, however, the tests are quite time consuming, expensive and not very simple.

The modulus of elasticity is often of concern, particularly for foundation problems in mudstones. The moduli vary from less than 2000 psi for overconsolidated clays, such as Bearpaw Shale, to 2×10^6 psi for well-cemented mudstones. Such a large variation of values is certainly desirable for classification and identification. The moduli,

however, depend on too many factors, such as sampling, discontinuities within the sample, degree of weathering, testing, interpretation of results, to provide unambiguous information.

The natural moisture content of mudstones varies from less than 5% upwards to 35% for some of the more porous sediments. The natural moisture content reflects the degree of induration of the sediments, but also indicates the state of weathering and deterioration. The moisture content is usually greater in weathered zones and broken zones than in zones of unweathered and intact material.

The permeability of mudstones is very low, and thus very difficult to determine. It is not well suited for classification purposes.

Swelling properties of mudstones are very important for the design of many engineering structures. Swelling is characteristic of all argillaceous sediments. It depends mainly on the percentage and type of clay minerals present. The determination of swelling properties is usually done by measuring the swelling pressure which develops when no volume change is allowed. These tests however are very time consuming and thus not very well suited as classification tests.

The Atterberg Limits have been found very useful for classification of fine grained soils. They also provide valuable information for the more soil like mudstones; however, for the harder,cemented materials the determination becomes more problematical and the results more ambiguous.

Grain size has been used successfully in the classsifica-

tion of medium to coarse grained soils and to some extent for the fine grained soils. The more indurated mudstones, however, have to be ground down for a grain size determination and therefore the results describe the powdered material rather than the original mudstone.

Most mudstones disintegrate readily upon cyclic drying and wetting. This disintegration process is generally called slaking. Several slaking tests have been established, which provide some quantitative information and can give a rough idea of how rapidly a shale will deteriorate on an exposed surface.

When mudstones are immersed in water they slowly lose some portion of their strength. Ordinary soils behave somewhat differently since they disintegrate rapidly when immersed in water. Based on these observations, a distinction between soil and rock has been suggested (Nakano, 1967).

A method for measuring water softening quantitatively appeared very attractive for an engineering classification as well as for an understanding of some aspects of progressive failure. Since water deterioration is an important characteristic engineering property, a quantitative study could well distinguish the more troublesome materials. Practical applications of such a study would be e.g.:

1) in longterm excavations, to predict strength changes and hence changes of stability with time

2) for surface durability considerations, e.g. of canal walls, above the water table as well as below the water table, or of tunnel walls
3) for the design of fills: selection of materials and degree of compaction
4) for the design of waste pits or tailing dams (resistance to weathering)
5) for the design of dams to find out how the fill materials change with time. This is important to know e.g. for the selection of materials for filter layers, or surface protection.

In addition a correlation between water softening and landslide activity for mudstones has been indicated (Nakano, 1967).

Hence tests were developed which measure the water deterioration of mudstones on the basis of

1) softening
2) slaking.

2. Water Deterioration Tests

2.1. Standard Compression Softening Test

It has been noted that mudstones remain intact when kept in a moist environment (e.g. Hamon and Post, 1969, for Shales from Java; Underwood, 1961, for Pierre Shale from South Dakota, USA) or when they are immersed in water at their natural water contents (Vaughan et al., 1967, for carboniferous shales from England; Nakano, 1967,

for Japanese mudstones; Nordquist et al., 1967, for Mancos Shale from Utah, USA). On the other hand, ordinary clay soils swell and disintegrate rapidly when placed into water at their natural water contents. Mudstones, which were immersed in water, were also found to swell slowly, decreasing in bulk density and strength (Nakano, 1967). The rate of reduction of the bulk density is, however, far slower than that for a clay soil. Once a mudstone is only slightly dried or mechanically disturbed, it disintegrates rapidly in water. Nakano (1967) found for several Japanese mudstones that drying in 94-97% humidity was enough to initiate disintegration. Balasubramonian (1972) squeezed pore water out of Bearpaw Shale by applying an allround pressure of 3000 to 4000 psi, thus reducing the natural water content from about 30% initially to about 26% after squeezing. When immersed in water the squeezed specimens readily fell apart. Based on the different water deterioration characteristics of clay soils and mudstones Nakano (1967) suggested the following definitions:

> "Clay soil is a material which is composed mainly of clay-fraction and is easily weakened, reducing its strength almost to zero, when the material at its natural water content is immersed in water. Mudstone is a material which in the same case keeps its original state and whose strength hardly decreases."

This classification is still rather crude and the limit between the two groups rather arbitrary. However, the idea of classifying and identifying according to the loss in strength with time is attractive, because:

1. the time dependent strength loss is a very significant and typical engineering property of overconsolidated clays and mudstones;
2. a differentiation between soils and rocks is important for many engineering design problems;
3. compression tests are simple to perform.

In order to develop a more refined classification based upon softening tests, the loss of compressive strength with time of immersion in water has been investigated for various materials ranging from overconsolidated clays to well-cemented siltstones.

2.1.1. Test Procedure

2.1.1.1. Sample Preparation

Samples were obtained from cores with diameters varying from 1.2 to 5.0 inches, and from block samples which were at their natural water content and were disturbed as little as possible. Cylindrical specimens were cut with the ratio of height to diameter equal to two.

Sample preparation at natural water content was done

a) by coring for hard, indurated mudstones,

b) by sawing and consequent sanding, when the material was too hard for carving and too little indurated for coring,

c) by carving with a knife, when the material was soft enough.

Sample sizes varied between 2.0 inches and 0.6 inches diameter, depending on the size of the original core, and on the fissility of the

material (See Table III.4.). Samples which were taken from cores with diameters of 1.0 to 2.0 inches were kept nearly unchanged in order to minimize disturbances. Core-samples with diameters of 3.0 to 5.0 inches were sawed into smaller pieces from which specimens of 1.0 to 1.5 inches diameter could be obtained when the samples stayed intact and did not fall apart. From materials which were fissile only rather small samples could be prepared (See Table III.4., Material No. 6). From very hard materials which had to be cored, specimens of 1.0 inch diameter were obtained. This size appeared convenient to avoid most joints and thus to obtain a maximum number of intact specimens.

Softened samples were prepared in the following manner:

Materials, in which no visible softening had occurred, were tested without further treatment. For materials, which had only slightly softened without visible change of shape, the surface zone was scratched off and the height to diameter ratio was controlled and corrected when necessary. When nonhomogeneous softening had occurred and the original shape had changed after parts of the sample had disintegrated, the samples were recarved with a knife in order to obtain regularly shaped specimens for the compression tests. During sample preparation disturbance was minimized by cautious handling and by excluding obviously disturbed zones within the material. Drying was minimized by preparing the samples in a moist room. This was not always possible, e.g. when the specimens had to be cored, sawed, and sanded. The samples were exposed as little as possible and protected with Sarane

wrap immediately after the preparation was finished and brought back to the moist room. During coring minimum drilling water was used.

The specimens were generally oriented in such a way that axial load was applied vertical to the bedding planes. Although intact samples were used during the tests, separation sometimes occurred along bedding planes. These discontinuities were found not to affect seriously the compression strength results under the chosen test conditions. However, the softening process was clearly accelerated by joints and fissures, and therefore broken specimens were not used for the softening stages. In order to obtain the most homogeneous and least disturbed samples a variation in sample size could not be avoided. It was found that variations in sample size did not affect the results more than other factors, which could not be controlled, such as variability of material from sample to sample and nonhomogeneities within the specimens. This can be seen by comparing Materials Nos. 1, 2, 3, and 4, which contain only specimens of constant size, with the remaining materials where variable sample sizes had to be used.

2.1.1.2. Testing

The samples were prepared at their natural water contents and were either tested immediately in a compression test at their natural water content or immersed in distilled water and after some duration of softening were tested in compression. The water content was determined for each material during sample preparation, and for each specimen after it had been tested in compression. The number of speci-

mens tested for each material depended mainly on the amount of material available and on sample loss during preparation. It was considered desirable to test at least 6 specimens at their natural water content and about 4 specimens of each at about 3 stages of softening (See Table III.4.).

a) Compression Test

The compression tests were generally performed at a confining pressure of 50 psi and a strain rate of 0.014 inches/min. The confining pressure of 50 psi was low enough to work with comfortably, but high enough to decrease substantially the influences of jointing and fissuring in soft and easily breakable materials. Even when dealing with badly broken materials, such as Bearpaw Shale, surprisingly consistent results could be obtained for comparable specimens (See Table III.4.). The specimens usually failed within 5 minutes for the hard, indurated materials and 15 minutes for the weaker materials. For some very strong materials, where the failure load exceeded the load capacity of the available testing arrangement, a hydraulic compression machine had to be used, where no confining pressure could be applied. However, the applied axial stresses of more than 1000 psi are very high compared to the confining pressure of 50 psi, so that it appeared permissible to compare these data with the remaining test results.

b) Immersion in Water

The specimens at their natural water contents were placed into glasses filled with distilled water, and left there for some time.

The time of water immersion was recorded for each specimen. The poorly indurated materials softened very quickly and lost about 90% of their original strength within a few hours. For these materials the loss in strength due to softening had to be tested at hourly intervals. For example, for Gault Clay (Sample No. 10) compression tests were performed on samples which had been immersed for 1.0, 2.5, 4.0, 4.5, 6.0, 7.0, 8.0, and 10.0 hours. Other more indurated materials showed measurable loss in strength only after days and approached a final strength value after weeks. Here it was sufficient to measure the strength loss due to softening at daily intervals. A third group of materials displayed a measurable strength loss only after weeks and reached a final softened stage after months. The strength loss was measured at weekly, or even monthly intervals. To find out the range of time for softening the softening study was started for each material with only a few samples. After it was clear whether the softening happened within hours, days, or weeks additional specimens could be conveniently added. The tested materials are described in detail in Tables Nos. III.1., III.2., III.3., III.4., and III.5.

2.2. Quantitative Slaking Test

The disintegration of mudstones upon alternate drying and wetting is generally called slaking. As noted previously Nakano (1967) observed that drying in 94% to 98% relative humidity is enough for some mudstones to start slaking and that mechanical disturbance also initiates slaking. Therefore it was concluded that mudstones may easily

deteriorate within the zone of fluctuating water vapor pressure or fluctuating groundwater level and within geologically fractured zones.

The slaking of dried mudstones has been explained by Terzaghi and Peck (1967) as being due to the tension produced by air in the voids of the dried mudstone. Nakano (1967) compared the slaking process of mudstones slaking in atmospheric pressure and of mudstones slaking in a vacuum. He found no substantial difference between the two cases and concluded that the main reason for slaking of dried mudstone is probably not the tension, produced by entrapped air, as Terzaghi and Peck had suggested, but some physico-chemical action. It also can be observed that mudstones containing large amounts of highly swelling clay minerals do slake more severely than mudstones which contain less swelling clay minerals. Because the physico-chemical properties are highly dependent on the surface area, it would appear that the degree of slaking, that a mudstone displays, is also reflected by its surface area. Thus surface area measurements would be a quantitative slaking index, too.

Surface area measurements are based on the property of clay minerals to attract bipolar molecules, such as nitrogen, ethylene glycol, glycerol, water, etc., to their surface (Brunauer, Emmet, Teller, 1938; Emmet, 1942; Brunauer, 1942 and 1943; Nelson and Hendricks, 1943; Mooney, Keenan, and Wood, 1952; Brooks, 1955; Diamond and Kinter, 1956; Shoemaker and Garland, 1967). Under certain conditions a monomolecular

layer will be absorbed to the clay surfaces. From the weight increase resulting from the absorbed molecules, the total surface area of the clay minerals can be computed. However, the standard methods of surface area measurements use powderized materials and cannot be applied to intact mudstones. At the outset of this study an attempt was made to develop a method by which the surface area of mudstone could be measured for intact materials and at various stages of slaking. Surface area measurements were made at first using the absorption of bipolar gases, with a molecule small enough to pass through the voids of the mudstone. The degree of absorption to clay surfaces could be determined by comparing the amount of gas retained for materials which had been exposed to bipolar gases and for materials which had been exposed to an inert gas, such as Helium. It was found, however, that for intact materials at room temperature the amount of gas absorbed on the clay surfaces was very small, when compared to the amount of gas retained within the pores of the samples. The procedure was abandoned in favour of absorption tests with water. Water molecules are highly attracted to clay minerals. However, the thickness of the absorbed water layer cannot be controlled easily, so that the surface area cannot be determined directly. Only relative values can be obtained. The increase of water content with the number of drying and wetting cycles was measured under controlled conditions. In spite of the relative crudeness of the test, the results obtained appeared promising, and thus the water-absorption test was adopted as a test for measuring quantitatively

slaking of clays and mudstones.

2.2.1. Test Procedure

2.2.1.1. Sample Preparation

Cylindrical samples were cut:

a) by coring, when hard, rock like mudstone

b) by carving with a knife, when softer, more clay like material was present

Three different sizes were tested:

1) 1.1" ø and 1.1" high

2) 1.4" ø and 1.4" high

3) 0.9" ø and 0.9" high

The majority of the tested samples had size No. 1 which appeared to be the most favourable since the specimens were big enough to include characteristic nonhomogeneities and small enough to obtain a number of specimens from available material.

The sample preparation, when done by carving with a knife, was a most delicate operation since most of the mudstones were brittle or fissured so that they broke easily during preparation. It appeared easiest to handle specimens of size No. 1. Water absorption and drying limited the upper size of the specimens. Since water absorption, to an equilibrium water content took 4 to 10 days for specimens of size No. 1 and about 14 days for specimens of size No. 2 it appeared unreasonable to investigate larger samples.

A lower limit was set by highly swelling soils due to the

often very substantial difference between water saturated volume and dry volume, the dry volume became very small so that the accuracy of the measurements was affected. Therefore smaller sizes of specimens than the ones tested were considered not to be reasonable.

2.2.1.2. Testing

The samples were alternately dried and wetted and the water content was taken after each wetting stage. 3 to 7 specimens were tested from each material.

a) Drying

The samples were dried for about 5 days at room temperature and for about 1 day in a desiccator at ca 72^{o} F and 18% humidity until an equilibrium water content was reached. The final water content was found to be about 4%. A few samples were ovendried. After drying the specimens were placed into fitting cylindrical, non-oxydizing containers, made out of brass, which had perforated walls and were open on both ends. Between the container walls and the sample some filterpaper was placed to allow a more uniform wetting of the samples and to prevent loss of sample through the walls of the containers. The purpose of these containers was to support the specimens, which often became very soft during the wetting process. The containers also provided more controlled conditions during the wetting process. One dimensional volume change could take place since horizontal movements were inhibited by the container. The containers were made out of screen to allow a better access of air during the drying process.

b) Wetting (See Fig. 3.13.)

It appeared necessary to take the dry weight of the specimens immediately before every wetting stage since during the drying process the tested material cracked and partly fell apart so that uncontrolled losses of material occurred. The samples were placed on wet filterpaper. The water was absorbed into the specimens quite readily until an equilibrium water content was reached after about 4 - 10 days for specimens of size No. 1. The test was performed in a moist room at 68^0 F and 90% humidity. The wetting process, which took very long for highly swelling materials after several slaking cycles, could be speeded up by turning the samples upside down at daily intervals.

To find the equilibrium water content it was sufficient to take the weight at the end of the wetting process. The volume change which occurred during the wetting process was often very substantial. To avoid uncontrolled loss of material, the soil which came out on both ends of the container was trimmed off and the decrease in weight was noted.

After the final water content was reached the specimens were dried again under the same conditions as described above.

2.3. Rate of Slaking Test

During the evaluation of the quantitative slaking test it was suggested that the rate of slaking during the very first slaking cycle might be relevant for classification. However, the slaking test as developed in the present study is too time consuming for such a

classification test. A complete cycle of drying and wetting takes at least 10 days. Therefore it was of interest to modify the test in order to measure the rate of slaking more rapidly. One way of accelerating the wetting stage is to immerse the dried specimens in water. However, for medium to highly slaking materials swelling and consequent disintegration occurs so quickly that it is difficult to control the process. This problem is overcome by immersing the samples in water after they had been placed into a funnel which was covered by filter-paper. The set-up is shown in Fig. 3.21.

Excess water drips off and the water content of the samples can be easily determined after various times of water immersion. Air-dried and ovendried samples were tested for a few materials, but no significantly different water contents after immersion could be detected (See Table III.7.). Because of the rather time consuming process of airdrying, this was abandoned and ovendrying was adapted as the standard procedure for this investigation. Tap water was used since no marked difference to distilled water was observed.

The water content was determined at various periods of water immersion. For most materials a constant water content was reached after less than 2 hours. An exception was Material No. 7, a highly bentonitc sandstone from the Edmonton Formation. Termination was obtained after about 24 hours of water immersion. The water content is plotted against time of water immersion for typical materials in Fig. A 3.62. Because termination was reached for most materials after 2 hours

of water immersion, this time-period was chosen as the standard time for this test which may be termed the rate of slaking test.

The standard procedure of the rate of slaking test is summarized in the following:

Irregular lumps of clays and mudstones smaller than 1 inch were dried to constant weight at around 105^{0} C. The dry weight of the samples was generally between 10 and 20 grams. The material was placed in a funnel which was covered by filterpaper, and then the material was immersed in water (See Fig. 3.21.). After 2 hours the funnel containing the samples was taken out, the excess water was allowed to drip off, and the water content was determined.

2.4. Test Results

2.4.1. General

A wide spectrum of materials was tested, materials which came from various parts of the world (Canada, U.S., England) and ranged from Cretaceous cemented siltstones from the Rocky Mountains to Tertiary overconsolidated clays (e.g. London Clay). In the strength softening test 14 different materials were tested (Materials No. 1 to 14 in Tables III.1., III.2., III.3.), in the slaking test 25 different materials (Materials No. 1 to 25 in Tables III.1., III.2., III.3.), and in the rate of slaking test 13 different materials (See Table III.7). The test results were correlated to standard classification tests, such as Atterberg Limits and grain size. In addition a mineralogical analysis of the clay portion was obtained. A systematic analysis of the carbonate

cementation was not undertaken, because the usual methods of carbonate determination are reported to be not very reliable and also because an analysis did not appear very fruitful, since no strong correlation was expected. Locker (1969) for example investigated a number of mudstones from Western Canada and reported very low carbonate contents for these materials. Many of those were tested in the present study (Materials Nos. 7, 8, 15, 16, 17, 18, 19, 20, 23), but independent of the generally low carbonate content widely varying degrees of water deterioration could be observed. For Jurassic sediments from England where limestones and clays were interbedded the carbonate content was found to be the same in both materials (Cruden, 1972). The substantial difference in properties and appearance between limestone and clay was probably caused mainly by the difference in clay content. Also, when comparing carbonate rich material, such as Oxford Clay (Material No. 9), with carbonate poor material, such as Bearpaw Shale (Material No. 8), no substantial difference in softening behavior could be observed. Oxford Clay contained an abundance of calcareous fossils and thus appeared as carbonate saturated, while for the Bearpaw Shale only very low carbonate content had been reported. The results of an analysis of the carbonate content can be very misleading when the carbonate in a mudstone is present in macro form, such as nodules or fossils, as for example in Oxford Clay.

An analysis of the silica-cementation would be very desirable, since a strong influence on the induration of materials is expected.

However, a dependable analysis is not possible so far for fine grained materials.

Liquid and Plastic Limit tests were performed according to ASTM designations D423-54T and D424-54T respectively. It has been argued (USCE, 1970) that the soil preparation according to ASTM standards (air-drying to constant weight, grinding and passing through No. 40 sieve) does not provide representative samples. Mechanical breaking of the dried material is believed to destroy the original grain structure of the soil and hence results, which are representative of the original soil, cannot be obtained. Three different procedures have been suggested, in which the materials are treated in a less severe manner (USCE, 1970). In a limited study, the influence of soil preparation on the Atterberg Limits of soils considered here was investigated.

Plastic Limit values were found to be essentially independent of soil preparation, but the Liquid Limit values were clearly influenced by the method of soil preparation. However, the deviation of these values from those found using the ASTM method did not exceed $\pm$ 20% (See Fig. A 3.65.). All three procedures as suggested by USCE are very time consuming and cannot be applied to materials, which are too hard to be carved with a knife, or which do not slake readily into a soft clay. Since a common procedure for all materials was considered more important than somewhat more representative results, the Limits according to ASTM standards were adopted in the correlations that follow.

Grain size determinations were performed by the hydrometer

technique in accordance with ASTM designation D422-54T, with the following modifications:

1) The samples were airdried and passed through a No. 40 sieve
2) The airdried soil powder was mixed with 10% Hydrogen-Peroxide-solution and boiled for about one hour in order to oxidize the organic portions.

The proportions of the clay minerals in the materials were determined from the clay content of the mudstones using X-ray diffraction methods. The X-ray analyses were performed at the Research Council of Alberta on oriented samples of the clay-sized fraction sedimented onto glass slides, which had been glycolized. The diffraction patterns were interpreted as suggested by Johns, Grim, and Bradley (1954).

2.4.2. Strength Softening

The results obtained from the strength softening tests generally show substantial scatter. This scatter reflects mainly the variation of material, structure, and jointing from specimen to specimen and to a lesser degree the influences of sample size or testing conditions (See Table III.4. and Figs. 3.1. and A 3.1. to A 3.13.). Only trends can be observed and for the further evaluation of the results this fact should be kept in mind.

For each material shear strength c_u has been plotted against time of softening during water immersion t_s (Figs. 3.1. and A 3.1. to A 3.13.). All materials show the same trend of strength decrease with increasing time of softening, with the strength eventually reaching a

final value, which is called the fully softened strength c_{uf}. For a better comparison of the different materials the strength values at each softening stage were normalized by referring the average softened strength values to the average initial strength: c_u/c_{uo} (Fig. 3.2.). In Fig. 3.3. the normalized shear strength c_u/c_{uo} has been plotted against time of softening on a logarithmic scale. The major portions of the strength softening curves are represented by straight lines, with the slopes varying slightly from material to material.

The time for loss of 50% of the original strength t_{50} was obtained from Fig. 3.3. and was plotted on a semilogarithmic graph against the normalized full strength loss $\Delta c_u/c_{uo}$ (Fig. 3.4.). In Fig 3.5. the normalized fully softened strength values c_{uf}/c_{uo} were plotted versus the time for full softening on a logarithmic scale. This shows that the materials which soften quickly also soften to a higher degree than materials which soften slowly.

Considering the rate of softening, three groups of materials can be separated. On the basis of time for full softening (Figs. 3.2., 3.3., 3.5.) they group into materials which are fully softened after

1) more than one month
2) within days
3) within hours

On the basis of 50% strength loss (Figs. 3.3. and 3.4.) they group into materials which

1) never lose 50% of their original strength

2) lose 50% of their original strength within days

3) lose 50% of their original strength within hours

A good correlation is found between normalized strength loss $\Delta c_u/c_{uo}$ and initial shear strength c_{uo} (Fig. 3.6.). Weak materials (c_{uo} < 250 psi) showed strength losses of more than 60% and strong materials (c_{uo} > 250 psi) indicated strength losses of less than 40%. From Fig. 3.6. as well as from Figs. 3.5., 3.8., and A 3.14. two groups of materials can be clearly differentiated on the basis of the amount of strength softening:

1) Materials which never lose more than 40% of their original strength

2) Materials which lose more than 60% of their original strength

Fig. A 3.14. shows that materials with higher natural water contents w_n soften more than materials with lower water contents. Only a rough correlation between natural water content w_n and initial strength c_{uo} is found, as shown in Fig. A 3.16.

The correlation between normalized strength loss $\Delta c_u/c_{uo}$ and percent clay size (Fig. A 3.26.) is very poor, only indicating that with increasing clay portions the strength loss increases. Better correlations are found when the normalized fully softened strength c_{uf}/c_{uo} is plotted against Activity A (Fig. A 3.27.) and against Plasticity Index I_p (Fig. A 3.39.). The fully softened strength c_{uf} decreases with

increasing A and increasing I_p. A poor relationship between normalized fully softened strength and initial Liquidity Index I_L is shown in Fig. A 3.25. The water contents were also recorded at each softening stage (Figs. A 3.17. to A 3.24.). These records represent time-swelling curves for natural materials in distilled water at zero load.

Fig. 3.7. indicates that there is a good correlation for all materials between normalized strength values c_u/c_{uo} during softening and increase of water content during softening Δw.

In Fig. 3.8. the normalized final strength loss $\Delta c_u/c_{uo}$ is plotted against final change of water content Δw_f for all materials. Materials with no measurable change in water content lose less than 40% of their original strength, and materials, where water content changes of more than 1% could be recorded, show a full strength loss of more than 60%. The undrained Moduli of Elasticity were determined from the compression test results as the slopes of the linear portions of the stress strain curves. This is illustrated in Fig. 3.11. In Fig. A 3.28. to A 3.38. the E-Moduli are plotted against the time of softening. The scatter of the results is often substantial. However, in most cases a general decrease of E-Modulus with time of softening can be observed, as would be anticipated.

Fig. 3.9. presents a linear correlation between undrained Modulus of Elasticity E and compressive strength c_u in a double logarithmic plot for all compression tests, on fresh as well as on softened specimens. Similar relationships had been found for Japanese

mudstones (Joshinaka, 1967) for rocks from Europe and North America (Deere and Miller, 1966) and for Pennsylvanian Shales from Illinois, U.S. (Hendron et al., 1970). These results are also indicated in the same graph. From Fig. 3.9. the average change of Modulus of Elasticity ΔE during time for 50% strength loss was taken at three representative strength values q_u and was plotted against the respective strength values in Fig. 3.10. A far larger decrease of stiffness is indicated for weak materials than for strong materials.

2.4.3. Slaking Test

The change of water content during wetting is indicated in Fig. 3.14. for Materials Nos. 7, 8c, 21, and 18 during their very first wetting stages. Material No. 7 is an example of a very highly slaking material, Material No. 8c is an example of a highly slaking material, Material No. 21 is an example of a medium slaking material, and Material No. 18 is an example of a slightly slaking material. The curves represent the swelling behavior of airdried natural materials under zero load with laterally restricted deformations. The equilibrium water content values w, which were obtained at the end of each wetting stage, were plotted against the number of drying-wetting cycles (Figs. 3.15., 3.16., and A 3.40. to A 3.54.). These plots show an increase of water content with number of drying-wetting cycles which is indicative of the slaking of the material. The water content values approach a maximum value w_s after about 6 to 12 cycles. The scatter of points in all these figures is due more to variation of material from specimen to specimen than to experimental deficiencies. For example,

Bearpaw Shale (Material No. 8c in Fig. 3.16.) shows two branches, which reflect the variation of materials, in this case also indicated by the variation of w_L and I_p values (See Table No. III.2.). A number of slaking tests were performed on remoulded specimens (Figs. A 3.40., A 3.47. A 3.48., A 3.49.). These specimens were obtained from the left-over material which had been prepared for the Atterberg Limit tests. A maximum water content was reached with the very first wetting stage. During the subsequent slaking cycles slightly lower water contents were obtained which were about the same as the maximum water content values for the natural samples. The maximum water contents w_s were plotted against Liquid Limit w_L (Fig. 3.17.): A linear increase of w_s with increasing w_L can be observed, indicating that during slaking all materials eventually reach a final water content equal to their Liquid Limits.

The increase in water content w* is the difference between maximum water content w_s and the original water content w_n. Values of w* have been plotted against Plasticity Index I_p in Fig. 3.18. A linear increase of w* with increasing I_p is found.

In Fig. 3.19. and in Fig. A 3.56. the change of Liquidity Index I_L with the number of drying-wetting cycles is described:

$$I_L = \frac{w - w_p}{I_p}$$

where w = final water content after each wetting stage

w_p = Plastic Limit

I_p = Plasticity Index

Here I_L is plotted against the square root of the number of drying--wetting cycles and straight lines can be drawn for the first 3 to 4 slaking cycles. The initial rate of slaking can be expressed by the slope of these lines. Thus the initial rate of slaking is given by the change of Liquidity Index I_L during the first slaking cycle:

$$\Delta I_{L1} = I_{L1} - I_{L0}$$

In Fig. 3.20. the Activity A is plotted against the maximum water content w_s. A band of values indicates a linear increase of w_s with A. Since Activity reflects the surface area of a mixture of several clay minerals, this plot can also be considered as a correlation between slaking and surface area. Fig. A 3.60. shows similarly that with increasing montmorillonite content an increase of maximum water content w_s can be expected.

There is no definite correlation between portion of clay sizes and maximum water content w_s.

The slaking test results w_s for the remoulded materials are plotted against w_L in Fig. A 3.61. and are compared with the correlation for undisturbed materials and with the correlation found for water absorption values after Enslin (Neumann, 1957). The water absorption from the slaking tests for the remoulded materials is higher than for the undisturbed samples, but its values are very close to those after Enslin.

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

2.4.4. Rate of Slaking Test

The results of the rate of slaking tests are summarized in Table III.7. In Fig. A 3.63. the water content after 2 hours of immersion is plotted against the Liquid Limit w_L and is compared with the water content values as obtained from the complete slaking tests as well as with those obtained after one slaking cycle. The water content after 2 hours of immersion increases with increasing Liquid Limit. Materials with low Liquid Limits mostly reach a water content which is below the value obtained for maximum slaking, but slightly higher than after one slaking cycle. For materials with high Liquid Limits the water contents after immersion mostly exceed the maximum slaking values. For materials with very high Liquid Limit values (w_L > 180%) the water contents after 2 hours of immersion are below the maximum slaking water content (Material No. 7). The maximum value, which can be obtained after 24 hours of water immersion for this material, again is higher than the maximum slaking value.

In Fig. A 3.64. the Liquidity Index after 2 hours water immersion is plotted against Liquid Limit and compared with the average values obtained from the quantitative slaking test.

In Fig. 3.22. the change of Liquidity Index ΔI_L during 2 hours immersion is plotted against Plasticity Index I_p and compared with the respective values as obtained from the slaking test. Both plots, the I_L - versus - w_L relationship as well as the ΔI_L - versus - I_p relationship, show a wide scatter of points and do not indicate a strong correlation.

2.5. Discussion of Results

2.5.1. Discussion of Strength Softening Results

On the basis of the results obtained the following observations can be made:

1) A good correlation exists between initial compressive strength and amount of strength loss during softening.
2) A distinction between two groups of materials can be made on the basis of strength loss during softening.
3) A differentiation of three groups of materials can be made on the basis of the rate of softening.
4) A correlation exists between compressive strength and undrained Modulus of Elasticity during all stages of softening.

The group of materials which showed only little strength reduction during water immersion (less than 40% of the original strength) were always characterized by undrained shear strength values higher than 250 psi. Most of these materials had low natural water contents ($w_n < 10\%$) and a small clay fraction ($\% < 2\mu < 15\%$). The only exception is Material No. 12 (Pierre Shale) which had a natural water content of 26% and a clay fraction of 35%. (It should be remembered that this is probably not typical for Pierre Shale.) For these materials the Plasticity Index I_p was generally less than 12% and the Activity always less than 0.35. In all cases no increase of water content during softening could be measured.

The second group of materials, which lost more than 60% of their original strength during water immersion, had a shear strength below 250 psi.

Sample No. 13 (Clagget Shale) did not follow this trend. In spite of an initial shear strength of 700 psi, it lost more than 80% of its original strength within 8 hours. The material, however, did not soften in the usual manner by a more or less homogeneous increase of water content throughout the sample. It fell apart along hidden joints and fissures. It can be assumed that the sample was not completely undisturbed. Disturbance could have been caused by

a) tectonic action (the core sample was densely jointed along bedding planes),

b) drying during storage (the sample protection was not intact when the core sample was received),

c) drying during sample preparation (the sample was sawed and consequently sanded outside of the moist room).

For all of the second group of materials natural water contents were higher than 10% and clay fractions were more than 25%; the Plasticity Index was above 24% and the Activity higher than 0.5. Generally an increase of water content during softening of more than 1% could be measured.

Three groups can be distinguished on the basis of the rate of softening. The first group of materials never loses more than 50% of its strength, and is identical to the previous first group of

little softening materials. The other two groups divide the softening materials into those which lose 50% of their original strength within days and into those which lose 50% of their original strength within hours.

The more rapidly softening materials generally show a strength loss which is slightly higher than for the slower softening materials. However, no further differentiation can be found on the basis of initial strength, E-Modulus, natural water content, change of water content during softening, clay size, Plasticity, or Activity. The group of slower softening materials includes e.g. Bearpaw Shale (Material No. 8) and Oxford Clay (Material No.9), while the more rapidly softening materials are represented by materials like Gault Clay (Material No. 10), London Clay (Material No. 11), or Colorado Shale (Material No. 14). The quoted examples indicate that materials which in North America are generally denoted as clay shales or compaction shales might be described in England as overconsolidated clays. London Clay is probably the best known example of a stiff fissured clay; therefore it appeared reasonable not to change the terminology for London Clay and to denote comparable materials in a similar manner. It was felt that the slower softening group of the materials that soften substantially is not essentially different from the more rapidly softening materials, and that therefore they should not be considered as a different material. They have been called hard clays, thereby making use of Terzaghi's classification "soft, medium, stiff, hard"

(Terzaghi and Peck, 1967). The group of materials which never lost more than 50% of their original strength during softening appeared far more indurated than the group of stiff and hard clays, and therefore more durable. This group should be considered from an engineering point of view as rocks and will be termed mudrocks or mudstones, or if fissile shales, which is consistent with the terminology suggested for inorganic, sedimentary material (Twenhofel, 1937; Underwood, 1967). On the basis of the strength softening test the following classification of argillaceous materials can be given:

1. Mudstones: $c_u > 250$ psi; strength loss during softening is $< 40\%$ of original strength
2. Clays : $c_u < 250$ psi; strength loss during softening is $> 60\%$ of original strength

2.1. Hard Clays: 50% strength loss occurs within days

2.2. Stiff Clays: 50% strength loss occurs within hours

2.3. Medium to Soft Clays: complete disintegration occurs immediately

Mudstones are further characterized by :

Modulus of Elasticity	E	$> 10\ 000$ psi
Plasticity Index	I_p	$< 12\%$
Activity	A	< 0.35
Clay fraction (mostly)	$(\% < 2\mu)$	$< 15\%$
Natural water content (mostly)	w_n	$< 10\%$
Maximum change of water content during softening	Δw	$\sim 0\%$

Clays are further characterized by:

Modulus of Elasticity	E	$< 20\,000$ psi
Clay fraction	($\% < 2\mu$)	$> 25\%$
Natural water content	w_n	$> 10\%$
Maximum change of water content during softening	Δw	$> 1\%$

2.5.2. Discussion of Slaking Test Results

The most important result from the slaking tests is the linear correlation between maximum water content w_s and Liquid Limit w_L: this indicates that during slaking all materials eventually reach water contents equal to their Liquid Limit values. The deviation of values from the w_s=w_L-line is generally small. Variations can be caused by not using identical samples for slaking tests and Atterberg Limit tests. Because of the small size of slaking specimens the slaking tests are far more sensitive to very small local variations of materials than the Atterberg tests. As large volume changes during the wetting stages took place, the material was sqeezed out of the slaking containers and had to be trimmed off. Small inclusions of highly swelling or non-swelling minerals within the zone of excess material consequently resulted in markedly lower or respectively higher water absorption values (See Material No. 7 in Fig. A 3.41. or Material No. 23 in Fig. A 3.54.).

A similar relationship to the w_s-w_L-correlation was found between water absorption ("Wasseraufnahme") after Enslin and Liquid Limit values (Neumann, 1957). The water absorption after Enslin

(Schultze and Muhs, 1950; Kezdi, 1964) is defined as the amount of water which can be absorbed be the fine portions of a dry soil (<0.06mm), and is expressed in terms of water content; the amount of absorbed water is measured in a specially designed apparatus, where the soil can suck in water through a filter. The Enslin water absorption values and the slaking values are compared in Fig. A 3.61. The two lines representing the respective correlations are parallel to each other but do not fall together. This deviation is mainly due to the difference in tested materials: the Enslin values are absorption values for the fine portions of a soil, while the slaking values are absorption values for the total material. It should be noted that the slaking values which were found for the remoulded materials (which had been passed through a No. 40 sieve, $\hat{=}$ 0.42mm) follow very closely the relation found for the Enslin-values, even though they generally contained coarser particles than the materials used in the Enslin test.

The slaking tests predicted little disintegration for materials with very low Liquid Limits (w_L < 20%). In the field, however, it was observed that materials which had only slightly disintegrated during the slaking tests completely disintegrated into a cohesionless silty or sandy mass after being exposed for less than one year (e.g. Materials Nos. 2 and 4). This suggests that for the very low plastic and non-plastic materials slaking is not the most destructive mechanism, and that cyclic freezing and thawing is probably the major agent of disintegration for these materials. Both mechanisms, however, support

one another. For materials with w_L > 50% slaking is a far more severe agent than frost action. During each wetting stage very substantial volume changes occur which cause large differential strains between different swelling particles and result in a complete disruption of the original structure.

Natural materials are likely bound together in clay clusters. The swelling properties are defined by the total surface area of the clusters and not of the actual clay mineral (Balasubramonian, 1972). During slaking these clusters break down, thus increasing the total surface area. Eventually, when the materials reach a water content equal to their Liquid Limit, the clusters can be considered as largely destroyed and finally the effective surface area is close to the surface area of the actual clay minerals. The correlation between maximum increase of water content and Activity (Fig. 3.20.), which is considered to be a good expression of surface area for a mixture of clay minerals, indicates this tendency. It is also interesting to compare the relationship between total surface area of the clay minerals and Plasticity or Liquid Limit, which has been observed for English clays (Farrar and Coleman, 1967) (See Figs. A 3.57. and A 3.58.). The relationships, which are very similar to the w_s-A correlation, indicate that the maximum water content after full slaking is a direct measure of the total surface area of the clay minerals. Similarly it is believed that the water content after each slaking stage reflects the respective effective surface area.

The influence of cementation on slaking can be recognized by comparing the slaking behavior of unweathered Bearpaw Shale (Material No. 8c) with weathered Bearpaw Shale (Material No. 8d) which was strongly ironstained (Fig. 3.16.). Results similar to the weathered Bearpaw Shale were obtained from unweathered Bearpaw Shale when slaked in iron containers and where ironstaining occurred during the test. The lower degree of slaking for the more cemented materials is also reflected by the decrease of Plasticity and Liquid Limit values with increase of cementation.

Material No. 8d was slaked in distilled water as well as in its porefluid (Balasubramonian, 1972), but no difference in water absorption could be observed for the two cases. It was concluded that salinity of the water to be absorbed had little influence on the slaking and hence swelling behavior of a soil. This observation is consistent with theoretical considerations on the swelling behavior of natural materials (Balasubramonian, 1972; Chattopadhyay, 1972).

The influence of ovendrying was studied during the first cycle for 4 samples (Materials Nos. 8d, 18, 24). Water absorption following ovendrying was markedly lower than for comparable airdried samples. Only after about 6 cycles were the same water absorption values reached as for the airdried samples.

The rate of slaking is best described by the change of Liquidity Index I_L with number of slaking cycles (Figs. 3.19. and A 3.56.). The relationship between I_L and number of cycles is linear

for the first 4 cycles in a square root plot. Some materials, however, show a steeper slope up to the first cycle than for the consequent points. It is interesting to note that these materials are either materials which could be classified in the strength softening test as stiff clays, or materials which had been weathered or remoulded. In this context it might be recalled that slaking tests on remoulded samples indicated maximum absorption values after the first slaking cycle (Figs. A 3.40., A 3.47., A 3.48., A 3.49.). Since the highest rate of slaking for some materials occurred during the first cycle, this rate was considered relevant for classification.

A classification according to maximum amount of slaking in terms of Liquid Limit (Fig. 3.17.) may be expressed as follows:

<u>Very low slaking</u> - the materials with $w_L < 20\%$; opening of fissures and only slight disintegration can be observed.

<u>Low slaking</u> - the materials with $20\% < w_L < 50\%$; the materials generally disintegrate into a granular, discontinuous mass.

<u>Medium slaking</u> - the materials with $50\% < w_L < 90\%$; the materials disintegrate into a medium soft clay, and often do not lose a granular structure.

<u>Highly slaking</u> - the materials with $90\% < w_L < 140\%$; the materials disintegrate into a soft clay of homogeneous appearance.

<u>Very highly slaking</u> - the materials with $w_L > 140\%$;

for these dominantly montmorillonitic materials it takes a very long time during each wetting stage until equilibrium water content can be reached (Fig. 3.14.), because of the very large volume change which takes place and the very low permeability. The materials lose their original structure during the first wetting stage.

2.5.3. Discussion of the Rate of Slaking Test Results

When comparing the water content values after water immersion with the slaking test results three groups of materials can be identified in Figs. 3.22. and A 3.64.:

1) Materials which reach water content values of the same magnitude as obtained during the slaking test after the first slaking cycle. These materials are generally of low plasticity with a Plasticity Index less than 20% and a Liquid Limit less than 50%.
2) Materials with water contents after immersion which are in the same range as the maximum slaking values. These materials soften more during water immersion than during the first slaking cycle but not more than is possible during the slaking test.
3) Materials which reach water contents which exceed the maximum slaking values. These materials soften substantially more than is possible during the slaking tests.

A classification according to the rate of slaking during 2 hours of water immersion may be expressed in terms of the change of Liquidity Index (See Fig. 3.22.) as follows:

Slow slaking - the materials with $\Delta I_L < 0.8$; these materials disintegrate into a fine to coarse grained gravel.

Fast slaking - the materials with $0.8 < \Delta I_L < 1.4$; these materials disintegrate into a soft homogeneous clay.

Very fast slaking - the materials with $\Delta I_L > 1.4$; these materials disintegrate into a very soft homogeneous mass.

A test similar to the rate of slaking test had been suggested by Hamrol (1961) for a "quantitative classification of the weathering and weatherability of rocks". Hamrol measured the water content of ovendried specimens after they had been immersed in water for about 1 1/2 hours and denoted it as the parameter i_I of weathering state I. He found that the weathering parameter correlated well with shearing resistance and Modulus of Elasticity of the rocks investigated.

The rate of slaking test supplements Hamrol's classification test. While Hamrol's test is concerned with the materials for which no Liquidity Index can be determined (for materials with $w_L \lesssim 20\%$), the rate of slaking test is applied to plastic materials. The classification of the plastic materials is based on the change of Liquidity Index during the period of water immersion rather than on the final wa-

ter content, because the change of Liquidity Index during a certain time period represents a more meaningful number.

Another test comparable to the rate of slaking test was developed by Franklin (1970). Franklin's test measures the disintegration of a soil after one or two slaking cycles.

In this test the samples are dried to constant weight at a temperature of 105^{0} C, placed into a drum, immersed in water and then rotated for about 10 minutes. The ratio of the amount of material left behind in the drum to the original amount of material was called the Slake Durability Index I_d and was used as a measure of slake durability.

Both tests measure the rate of slaking of ovendried specimens after immersion in water. The rate of slaking test determines the amount of disintegration by measuring the water content after water immersion, while Franklin's slake durability test determines the amount of disintegration after water immersion and mechanical disturbance on the basis of grain size. The wetting stage of the rate of slaking test is substantially longer than in Franklin's test (2 hours against 10 minutes), however, in Franklin's test the materials are also affected by a mechanical process. It can be assumed that both tests provide similar results. A direct comparison was obtained only for Material No. 9 (Oxford Clay) and Material No. 13 (Clagget Shale). According to Franklin's test Oxford Clay is a medium to high durable clay and Clagget Shale a very little durable clay (Gamble, 1971). According to the rate of slaking test Oxford Clay is termed a slow

slaking clay and Clagget Shale a fast slaking mudstone. A comparison over a range of index properties is not possible since both tests show only very crude correlations with index properties, which in addition appear insensitive (Gamble, 1971). Franklin's test results by themselves are not very meaningful, because the dynamic test conditions, under which the tests have been run, are basically different from natural conditions under which slaking occurs. Franklin's test data, however, correlate different materials to each other based on the amount of disintegration by the specific test conditions.

The rate of slaking test as developed in this study is comparable to natural conditions and the results can be expressed in terms of Liquidity Index, which is a physically meaningful index number. In addition the rate of slaking test results can be easily compared with the results of the quantitative slaking test, which obviously indicates basic soil properties.

From this point of view the rate of slaking test appears clearly advantageous as compared with Franklin's slake durability test. Besides, the necessary testing equipment for the rate of slaking test is cheaper and less complicated than for Franklin's test.

2.5.4. Proposal of a Classification System

A classification system for inorganic noncalcareous sedimentary material is proposed on the basis of strength softening results as indicated in Fig. 3.23.

According to the quantitative slaking test and the rate of slaking test the inorganic sedimentary materials can be further identified and designated as shown in Fig. 3.24.

TABLE III.1.

WATER DETERIORATION TESTS: SUMMARY OF MATERIALS TESTED

AND THEIR QUALITATIVE DESCRIPTION

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens*	Depth Below Surface in feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
1	Waipi Ft.	Foothills at Big Horn Dam, Alberta, Canada	light grey	a	300	no visible change	no visible change	Sandstone L	intact, well cemented, include thin dark bands
2	Waipi Ft.	Foothills at Big Horn Dam, Alberta, Canada	dark grey	a	300	no visible change	disintegration into flakes	Shale L	fissured, include thin bands of light grey materials
3	Waipi Ft.	Foothills at Hinton, Alberta, Canada	yellow	a	70	no visible change	no visible change	Sandstone L	intact, cemented
4	Waipi Ft.	Foothills at Hinton, Alberta, Canada	light grey	a	70	little change visible	some softening and disintegration	Sandstone L	intact, coarse grained, little cemented, black organic inclusions, occasionally ironstained

* a: coring -- hard c: carving possible -- medium soft
b: sanding -- medium hard d: carving easy -- soft

TABLE III.1. Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
5	Battle F. of Upper Cretaceous	Cyprus Hills, Alberta Canada	light blue-grey	c	60	thin, smeary soft zone on surface	change into soft whitish clay with granular structure	Hard Clay L to M F	break easily along bedding plane and joints vertical to it
6	Battle F. of Upper Cretaceous	Cyprus Hills, Alberta Canada	medium brown	c	60	show softened surface zone	after one cycle disintegration into fine gravel	Hard Clay L to M S	strongly fissured, jointed and slickensided, very brittle
7	Edmonton F. of Upper Cretaceous	Edmonton Alberta Canada	medium grey	d	15	soft, smeary zone on surface	after one cycle change into soft clay - extremely large swelling	Hard to Stiff Clay VH S to F	intact, homogeneous

TABLE III.1. Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
8	Bearpaw F. of Upper Cretaceous	Cyprus Hills, Alberta, Canada	dark to medium grey-brown	c	200 to 1000	soft surface zone; occasionally considerable softening along joints	change into soft clay of lumpy structure	Hard Clay M F	intact, varying from sandy to silty material
8a			medium to dark grey-brown	c	560		change into soft clay of lumpy structure	M —	intact, clayey
8b				c	225			L —	intact, sandy-silty
8c		South Saskatchewan Dam, Saskatchewan, Canada		c	12			H —	strongly fissured and slickensided; very brittle

TABLE III.1 Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
8d	Bearpaw F. of Upper Cretaceous	South Saskatchewan Dam, Saskatchewan, Canada	medium to dark grey-brown	c	4		change into soft clay of lumpy structure	M —	strongly fissured and slickensided; strongly weathered, ironstained along joints
9	Oxford Clay Upper Jurassic	England	dark to medium brown	c		softened surface zone	soften but no visible distinction	Hard Clay L S	contains abundance of calcareous fossils, break easily along bedding planes
10	Gault Clay Upper Cretaceous	England	blueish-grey	d		disintegration along bands of darker materials	change into soft clay	Stiff Clay M S to F	darker and lighter materials are intermixed; fissured and jointed; along fissures ironstained
11	Brown London Clay Eocene	England	yellowish brown	d	6.2	visible disintegration on surface	change into soft clay	Stiff Clay M —	contains open joints; along joints stronly ironstained

TABLE III.1 Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
12	Pierre F. of Upper Cretaceous	Upper Missouri River, Montana, U.S.	brownish grey	c	85	no visible change	separate along joints and partly disintegrate to silt	Mudstone L to M S	very homogenous, silty, not typical for Pierre Formation
13	Clagget F. of Upper Cretaceous		dark grey to medium grey	b	450	separation along fissures	change into very soft clay	Mudstone to Hard Clay M to H VF	strongly jointed along bedding planes
14	Colorado F. of Upper Cretaceous		medium grey to dark brown	d	40	very rapid softening and disintegration	change very fast into soft clay	Stiff Clay M	silty medium grey and dark brown carbonaceous materials are interbedded
15	Lowermost Paskapoo Paleocene	Entwistle Alberta, Canada	medium grey	b	114		change into soft clay	M S to F	fossils along bedding planes, brittle; contains fine coal fragments

TABLE III.1 Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
16	Lowermost Paskapoo Paleocene	Entwistle, Alberta, Canada	light grey	d	50		change into soft clay	$\underline{M}$	interbedding of material No. 16 and No. 17. contains coal fragments
17				b to c	51		fissuring after 3 cycles and consequent some disintegration	$\underline{L}$	
18	Marlboro F. of Upper Cretaceous	McLeod Riber, Alberta, Canada	medium grey to yellow	a	114		after 5th cycle disintegration into fine gravel	L S - F	generally homogeneous, iron-stained zones appear softer and more clayey
19	Saunders Group Paleocene	Shining Bank Ridge, Alberta, Canada	grey to brownish grey	c	1		grey materials change into medium soft granular soil brown materials into soft clay	$\underline{M}$	interbedding of grey hard fissured shale and brown softer clay, iron-stained along joints

TABLE III.1 Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
20	Belly River F. of Upper Cretaceous	Waskatenau, Alberta, Canada	greyish brown	c	39		change to soft clay along coal rapid softening	M —	intact, contains many coal fragments
21	Dunvegon F. of Upper Cretaceous	Dunvegon, Alberta, Canada	dark greyish brown	d	15		change into soft clay	H —	jointed and fissured
22	Edmonton F. of Upper Cretaceous	Wabamun, Alberta, Canada	dark brown	c	10		change into granular soft material	M VF	very brittle, strongly fissured, highly organic
23		Edmonton, Alberta, Canada	medium grey	d	35		change into soft clay	H —	generally homogeneous with a few inclusions of whitish concretions

TABLE III.1 Continued

Material No.	Geological Formation	Locality	Colour	Preparation of Specimens	Depth Below Surface in Feet	Description of Softening	Description of Slaking	Classification and Identification according to Water Deterioration Test	Additional Remarks
24	Glacial Till of Pleistocene	Edmonton, Alberta, Canada	dark to medium grey-brown	c	40		change into soft silt	L VF	contains quartz pebbles up to 1cm. diameter, jointed, joint surfaces are ironstained
25	Kootenay F. of Upper Cretaceous	Crowsnest Pass, Alberta, Canada	dark brown	a	200		upon first drying disintegration into flakes no further visual changes	VL —	strongly jointed and fissured
26	Edmonton F. of Upper Cretaceous	Devon, Alberta, Canada	medium grey	c	40		changes into soft silty clay	L —	naturally weathered specimen

TABLE III.2.

WATER DETERIORATION TESTS: SUMMARY OF INDEX PROPERTIES

Material No.	$\% < 2\mu$	w_N	w_p	w_L	I_p	$A = \frac{I_p}{\% < 2\mu}$	$I_L = \frac{w_N - w_p}{I_p}$	Min. An.
	%	%	%	%	%			
1	~ 0	1.5	--	--	--	→ 0	--	yes
2	13	1.7	17	17.5	0.5	.04	- 30	yes
3	10	5	--	18.9	--	→ 0	--	yes
4	8	7.2	--	20	--	→ 0	--	yes
5	55	16.7	21.1	50.7	29.5	.55	- .15	yes
6	25	21.2	25.7	50.0	24.3	.97	- .18	yes
7	30	17.2	17	~180	~163	5	± 0.0	yes
8a	47	23.1	25	76	51	1	- .04	no
8b	30	18.2	17.2	47.2	30	1	+ .03	no
8c	48	28	32	127-104	94-74	1.8	- .06	yes
8d	44	25	28.6	62.1	35.5	.76	- .11	yes
9	--	20.2	18.4	47.3	28.9	--	+ .06	yes

TABLE III.2. Continued

Material No.	$\% < 2\mu$	w_N	w_p	w_L	I_p	$A = \frac{I_p}{\% < 2\mu}$	$I_L = \frac{w_N - w_p}{I_p}$	Min. An.
	%	%	%	%	%			
10	83	21.9	24.9	68.5	43.6	.53	- .07	yes
11	55	26	29.8	72	42.2	.77	- .09	yes
12	35	25.8	32.7	44.7	12	.35	- .2	no
13	33	13.5	27.8	89.8	62	1.88	- .23	no
14	34	28.5	27.8	68.0	40.2	1.18	+ .02	no
15	45	10	17.2	56.5	39.3	.89	- .18	yes
16	40	18	18.5	54	35.5	.89	- .14	yes
17	11	13	27	34	7	.66	-1.71	yes
18	15	9	21	31.6	10.6	.71	-1.13	yes
19	35	11	18.1	57.5	39.4	1.13	- .18	yes
20	36	15	20.1	74.2	54.1	1.5	- .09	yes
21	45	28	30	45	15	.33	- .13	yes
22	50	25	33.4	57.3	23.9	.48	- .34	no

TABLE III.2. Continued

Material No.	%<2μ	w_N	w_p	w_L	I_p	$A = \frac{I_p}{\%<2\mu}$	$I_L = \frac{w_N - w_p}{I_p}$	Min. An.
	%	%	%	%	%			
23	53	28	32.8	10.7	74.2	1.4	- .06	yes
24	22	14	15.8	31.9	16.1	.73	- .11	no
25	--	4	--	--	--	--	--	no
26	28	26.6	23	44	21	.75	+ .11	no

TABLE III.3.

SUMMARY OF MINERALOGICAL DATA

Material No.	Montmorillonite	Illite	Kaolinite	Montmorillonite	Illite	Kaolinite	Carbonate Content
	(% of clay-fraction) (after Research Council of Alberta)			(% of total weight)			(Locker, 1969)
1	0	95	5	0	1	0	
2	0	95	5	0	12	0	
3	55	30	15	6	3	2	
4	60	20	20	5	2	2	
5	15	65	20	8	36	11	
6	35	45	20	9	11	5	
7	100	0	0	30	0	0	0
8a	--	--	--	--	--	--	
8b	--	--	--	--	--	--	
8c	55	40	5	26	18	2	
8d	35	55	10	15	24	4	
9	5	65	30	--	--	--	
10	35	45	20	29	37	17	
11	20	50	30	11	27	17	
12	--	--	--	--	--	--	
13	--	--	--	--	--	--	
14	--	--	--	--	--	--	
15	25	55	15	11	25	7	0
16	70	30	0	28	12	0	0
17	45	20	32	5	2	3	1.5

TABLE III.3. Continued

Material No.	Montmorillonite	Illite	Kaolinite	Montmorillonite	Illite	Kaolinite	Carbonate Content
	(% of clay-fraction) (after Research Council of Alberta)			(% of total weight)			(Locker, 1969)
18	--	--	--	6	7	0	0
19	--	--	--	8	26	0	
20	--	--	--	14	11	4	
21	2	66	33	1	30	15	
22	--	--	--	--	--	--	
23	75	20	5	40	10	3	
24	--	--	--	--	--	--	
25	--	--	--	--	--	--	
26	--	--	--	--	--	--	

TABLE III.4.

STANDARD COMPRESSION SOFTENING TEST: SUMMARY OF TEST

CONDITIONS AND VARIATION OF STRENGTH DATA

Material No.	No. of Specimens	Diameter of Specimens		Confining Pressure	Maximum Deviation of Shearstrength Cu from Medium Value: ΔCu		Normalized Deviation $\Delta Cuo/Cu$	
		Fresh (inch)	Softened (inch)	(psi)	Fresh (psi)	Softened (psi)	Fresh	Softened
1	31	1.0	1.0	0	4000	4000	.45	.50
2	15	1.0	1.0	0	700	300	.33	.23
3	22	1.0	1.0	0	11500	600	.72	.55
4	38	1.0	1.0	0	160	110	.45	.46
5	20	.9 to 1.9	.7 to 1.4	50	24	12	.39	.57
6	24	.6 to 1.1	.6 to .83	50	28	19	.38	.64
7	20	.94 to 1.5	.78 to 1.5	50	85	0.5	1.5	.33
8	23	1.45 to 2.1	1.25 to 2.1	50	210	5	.88	.62
9	25	1.05 to 1.24	.8 to 1.27	50	38	17	.54	.89
10	33	1.5	.75 to 1.1	50	4	--	.25	--

TABLE III.4. Continued

Material No.	No of Specimens	Diameter of Specimens Fresh (inch)	Diameter of Specimens Softened (inch)	Confining Pressure (psi)	Maximum Deviation of Shearstrength Cu from Medium Value: ΔCu Fresh (psi)	Maximum Deviation of Shearstrength Cu from Medium Value: ΔCu Softened (psi)	Normalized Deviation $\Delta Cuo/Cu$ Fresh	Normalized Deviation $\Delta Cuo/Cu$ Softened
11	13	1.0 to 1.69	1.0 to 1.45	50	7	4	.17	.67
12	10	.85 to 1.20	1.05 to 1.65	50	40	80	.13	.32
13	9	1.65 to 1.20	.87 to 1.20	50	300	0	.50	0
14	8	1.20 to 1.82	1.0	50	16	3	.55	.60

TABLE III.5.

STANDARD COMPRESSION SOFTENING TEST: SUMMARY OF RESULTS

Material No.	w_N (%)	w_{final} (%)	Δw (%)	Eo x 10^3 (psi)	Ef x 10^3 (psi)	Cuo (psi)	Cuf (psi)	ΔCu (psi)	$\frac{Cu}{Cuo}$	$\frac{Cuf}{Cuo}$	Time for Full Softening (days)	Time for 50% Softening (days)	Classification
1	1.5	1.5	0	--	--	9000	8000	1000	.11	.89	175	∞	Sandstone
2	1.7	1.7	0	300.0		2100	1300	800	.38	.62	170	∞	Mudstone
3	5.0	5.0	0	80.0	80.0	1600	1100	50	.31	.69	40	∞	Sandstone
4	7.2	7.2	0	20.0	20.0	360	240	12	.33	.67	35	∞	Sandstone
5	16.7	20.7	4.0	3.1	1.2	61	21	40	.66	.34	7	3.5	Hard Clay
6	21.2	23.1	1.9	3.0	3.0	73	30	43	.59	.41	24	13.0	Hard Clay
7	17.2to27.0		8.0	6.0	1.5	55	15	40	.85	.15	4	.5	Hard to Stiff Clay
8	17.6to29.0		5.0	20.0	10.0	240	80	160	.65	.35	8	4.5	Hard Clay
9	20.2to24.4		5.0	3.6	.8	70	19	51	.73	.27	15	4.5	Hard Clay
10	21.9to38.2		11.0	1.2	1.2	16	0	15	1.0	0	.33	.13	Stiff Clay
11	26.0to31.8		5.0	3.1	.5	40	6.0	34	.85	.15	.08	.02	Stiff Clay
12	25.8to28.4		0	24.0	24.0	300	250	50	.17	.83	25		Mudstone

TABLE III.5. Continued

Material No.	w_N (%)	w_{final} (%)	Δw (%)	Eo x10^3 (psi)	Ef x 10^3 (psi)	Cuo (psi)	Cuf (psi)	ΔCu (psi)	$\frac{\Delta Cu}{Cuo}$	$\frac{Cuf}{Cuo}$	Time for Full Softening (days)	Time for 50% Softening (days)	Classification
13	13.5to15.0		1.5	150.0	20.0	600	100	50	.83	.17	.29	.08	Mudstone to Hard Clay
14	28.5to42.0		15.6	1.2	.2	29	5.0	24	.83	.17	.22	.05	Stiff Clay

TABLE III.6.

QUANTITATIVE SLAKING TEST: SUMMARY OF RESULTS

Material No.	w_N (%)	w_L (%)	I_p (%)	w_s (%)	$w^* = w_s - w_N$ (%)	w_s remoulded (%)	Classification according to amount of Slaking
1	1.5	--	--	2.5	1.0	--	VL
2	1.7	17.5	.5	6.5	4.8	35	VL
3	5.0	18.9	--	13.0	6.0	39	VL
4	7.2	20.0	--	19.0	19.0	36	VL to L
5	21.0	50.7	29.6	46.0	22.0	--	L to M
6	21.2	50.0	24.3	46.0	22.0	--	L to M
7	17.2	180.0	163.0	200.0	180.0	--	VH
8a	23.1	76.0	51.0	90.0	57.0	--	M
8b	18.2	47.2	30.0	43.0	24.8	--	L
8c	28.0	115.0	83.0	115.0	87.0	--	H
8d	25.0	62.1	33.5	55.0	30.0	--	M
9	20.2	47.3	28.9	42.0	22.0	--	L

TABLE III.6. Continued

Material No.	w_N (%)	w_L (%)	I_p (%)	w_s (%)	$w^* = w_s - w_N$ (%)	w_s remoulded (%)	Classification according to amount of Slaking
10	21.9	68.5	43.6	60.0	38.0	--	M
11	26.0	72.0	42.2	70.0	44.0	--	M
12	26.0	44.7	12.0	42.0	16.0	--	L to M
13	13.5	89.8	62.0	100.0	86.5	--	M to H
14	28.5	68.0	40.2	80.0	51.5	--	M
15	10.0	56.5	39.3	53.0	43.0	65.0	M
16	18.0	54.0	35.5	70.0	52.0	73.0	M
17	13.0	34.0	7.0	26.0	13.0	--	L
18	9.0	31.6	10.6	25.0	15.0	47.0	L
19	11.0	57.5	39.4	67.0	56.0	--	M
20	15.0	74.2	54.1	90.0	75.0	--	M
21	28.0	45.0	15.0	46.0	18.0	--	L
22	25.0	57.3	23.9	52.0	27.0	--	M

TABLE III.6. Continued

Material No.	w_N (%)	w_L (%)	I_p (%)	w_S (%)	$w^* = w_S - w_N$ (%)	w_S remoulded (%)	Classification according to amount of Slaking
23	28.0	107.0	74.2	122.0	94.0	--	H
24	14.0	31.9	16.1	24.0	10.0	--	L
25	2.0	--	--	6.0	4.0	--	VL
26	26.6	44.0	21.0	50.0	23.4	--	L

TABLE III.7.

RATE OF SLAKING TEST: SUMMARY OF RESULTS

Material No.	w_N	w_L	I_p	I_{LO}	w_I				I_{LI}		ΔI_L		Classification according to Rate of Slaking
					1st cycle of slaking	Airdrying +2h of water immersion	Ovendrying + 2h of water immersion range	Ovendrying + 2h of water immersion average	1st Cycle of Slaking	Rate of Slaking Test	1st Cycle of Slaking	Rate of Slaking Test	
	(%)	(%)	(%)		(%)								
4	7.2	20.0	--	--	15.0	--	4.1 - 10.2	7.0	--	--	--	--	S
5	16.7	50.7	29.6	- .19	32.0	--	42.2 - 57.0	49.7	.33	.93	.52	1.12	F
6	21.2	50.0	24.3	- .09	32.0	--	34.5	34.5	.25	.36	.34	.45	S
7a	17.2	220.0	202.8	0	180.0	--	166.0 -183.0	175.0	.81	.78	.81	.78	S to F
7b	17.5	200.0	182.5	0	150.0	126.0	86.0 -117.0	105.7	.74	.49	.74	.49	S
8	28.0	115.0	94.0	- .01	57.0	91.5	137.0	137.0	.26	1.11	.26	1.12	F
9	20.2	47.3	28.9	+ .06	33.0	--	23.6 - 30.1	27.8	.50	.33	.44	.27	S
10	21.9	68.5	43.6	- .09	48.0	--	45.1 - 61.0	54.3	.44	.67	.53	.76	S to F
12	25.8	44.2	12.0	- .20	37.0	--	25.0 - 25.4	25.2	.33	.17	.53	.37	S
13	13.5	89.9	62.0	- .23	49.0	109.0	129.0 -169.0	149.0	.34	1.95	.57	2.18	VF
15	10.0	56.5	40.0	- .18	30.0	--	31.8 - 49.0	40.4	.32	.58	.62	.76	S to F
18	9.0	31.6	11.0	- 1.15	15.0	33.6	12.8 - 30.0	26.5	- .54	- .50 - +.50	.69	.65-1.65	S to F
22	25.0	57.3	23.9	0	47.0	--	102.5	102.5	.25 - 1.34	2.90	.25-1.34	2.90	V F
24	14.0	31.9	16.1	- .11	20.0	43.4	37.2	37.2	.25	1.30	.45	1.41	VF

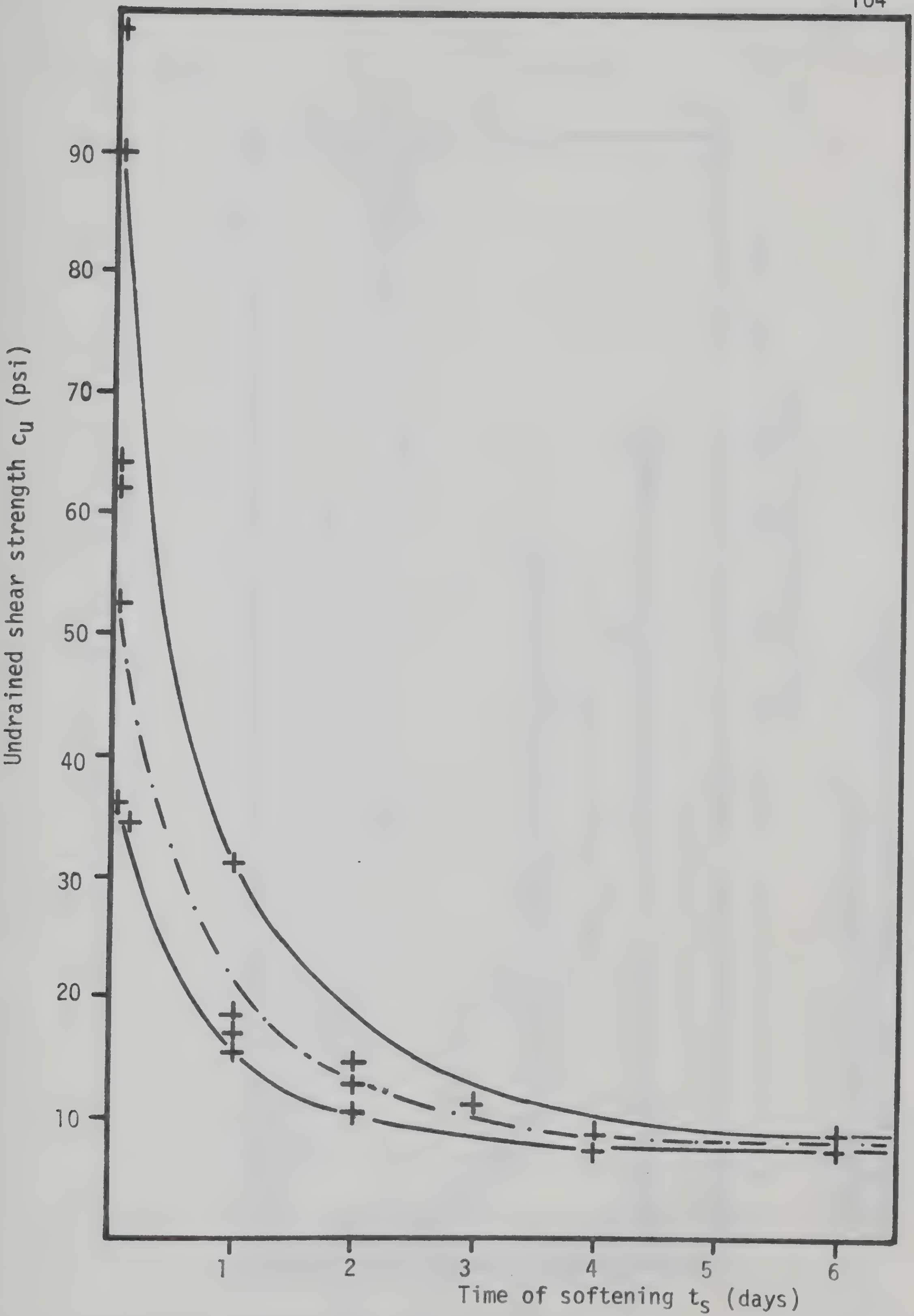
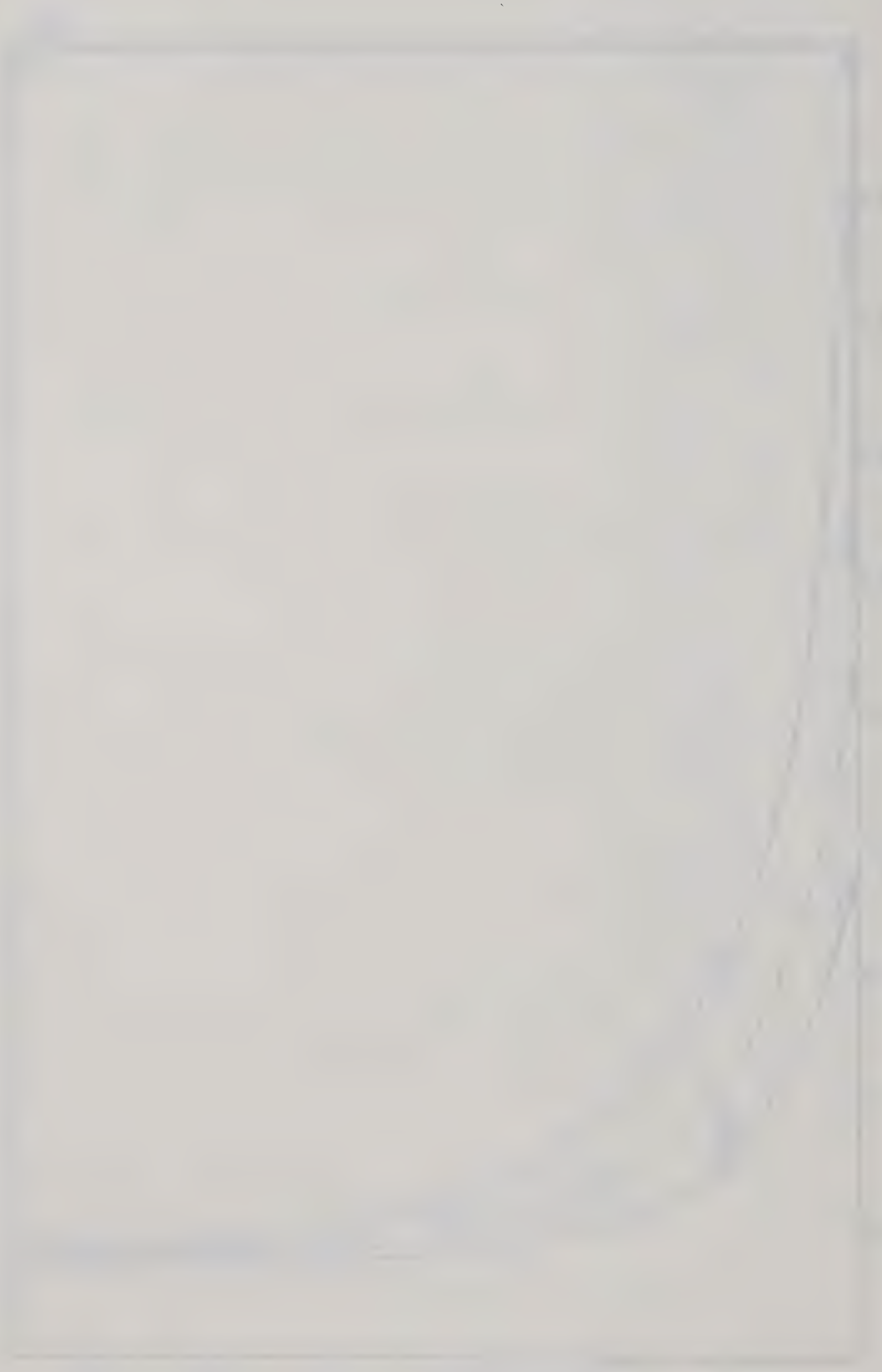


FIG. 3.1. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF SOFTENING FOR MAT. NO.7



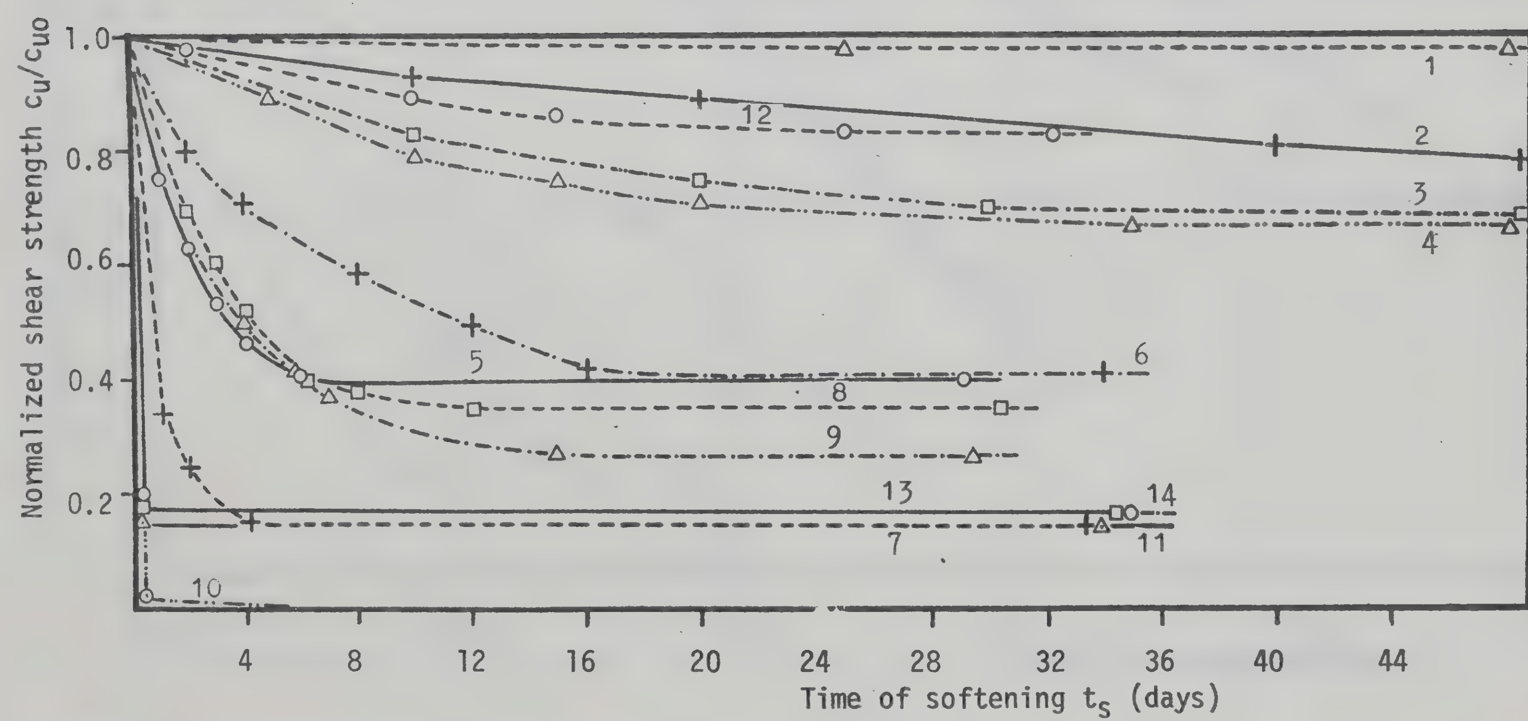


FIG. 3.2. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED STRENGTH VERSUS TIME OF WATER IMMERSION.

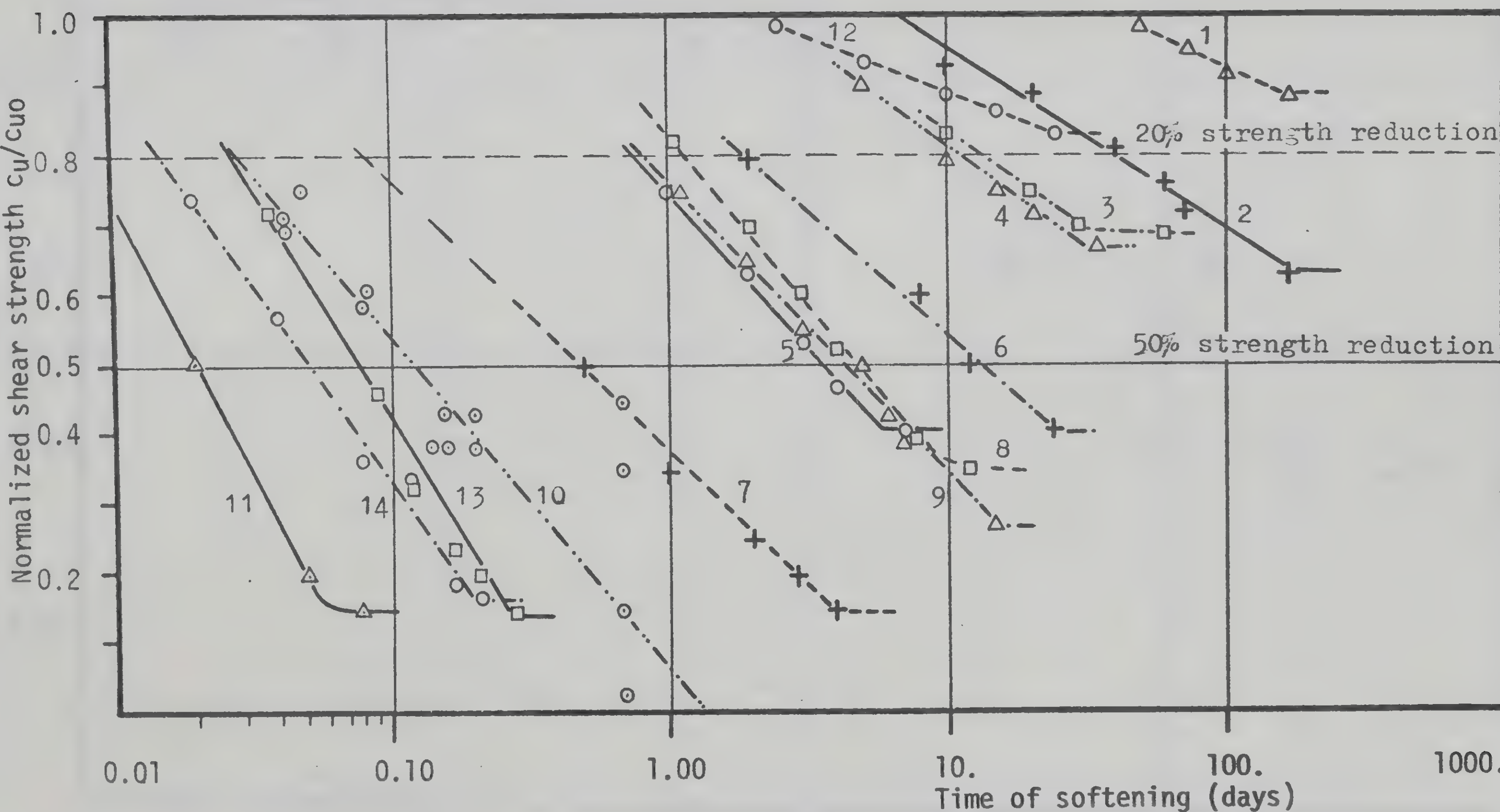


FIG. 3.3. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED STRENGTH VERSUS TIME OF SOFTENING.

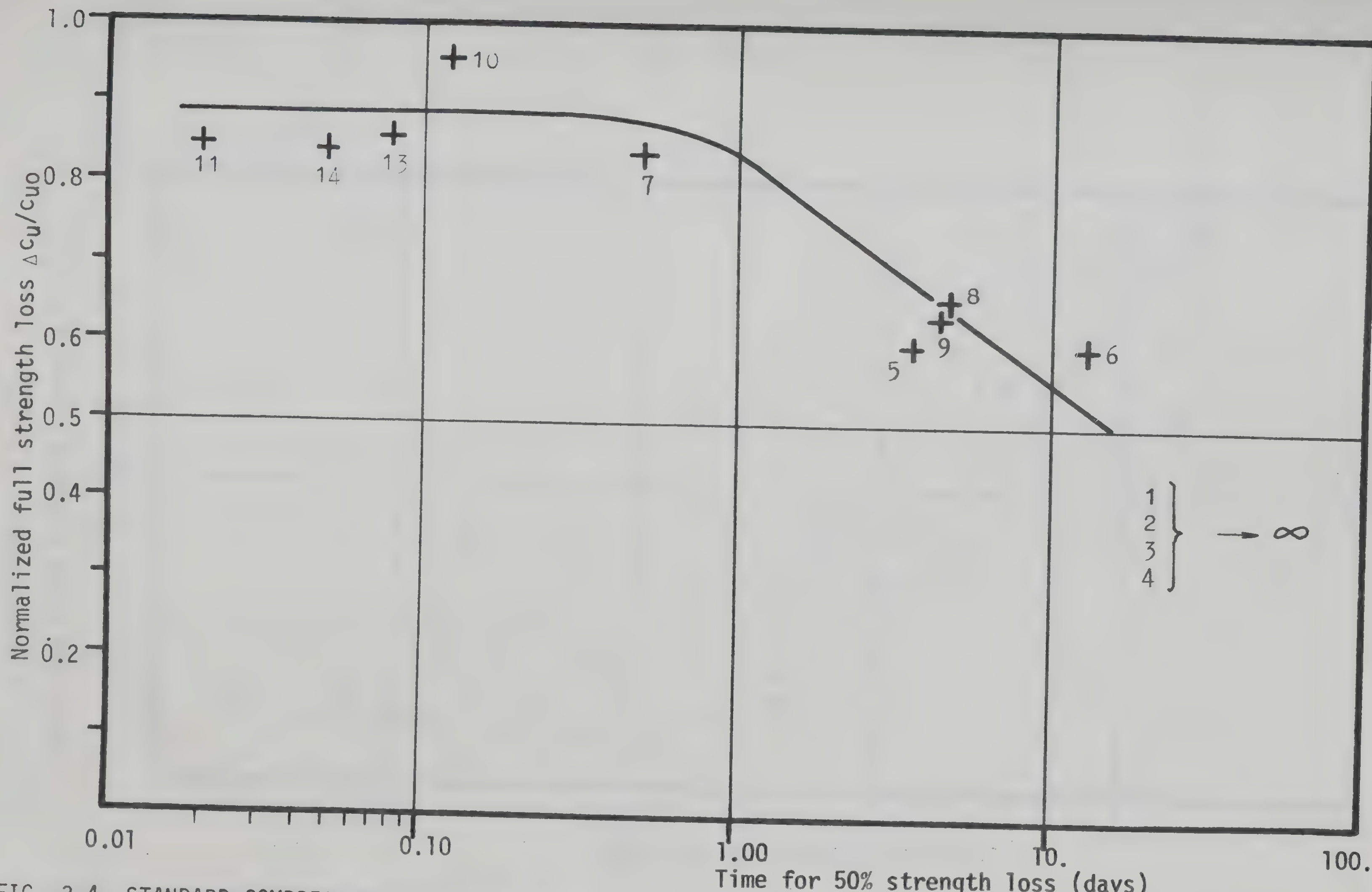
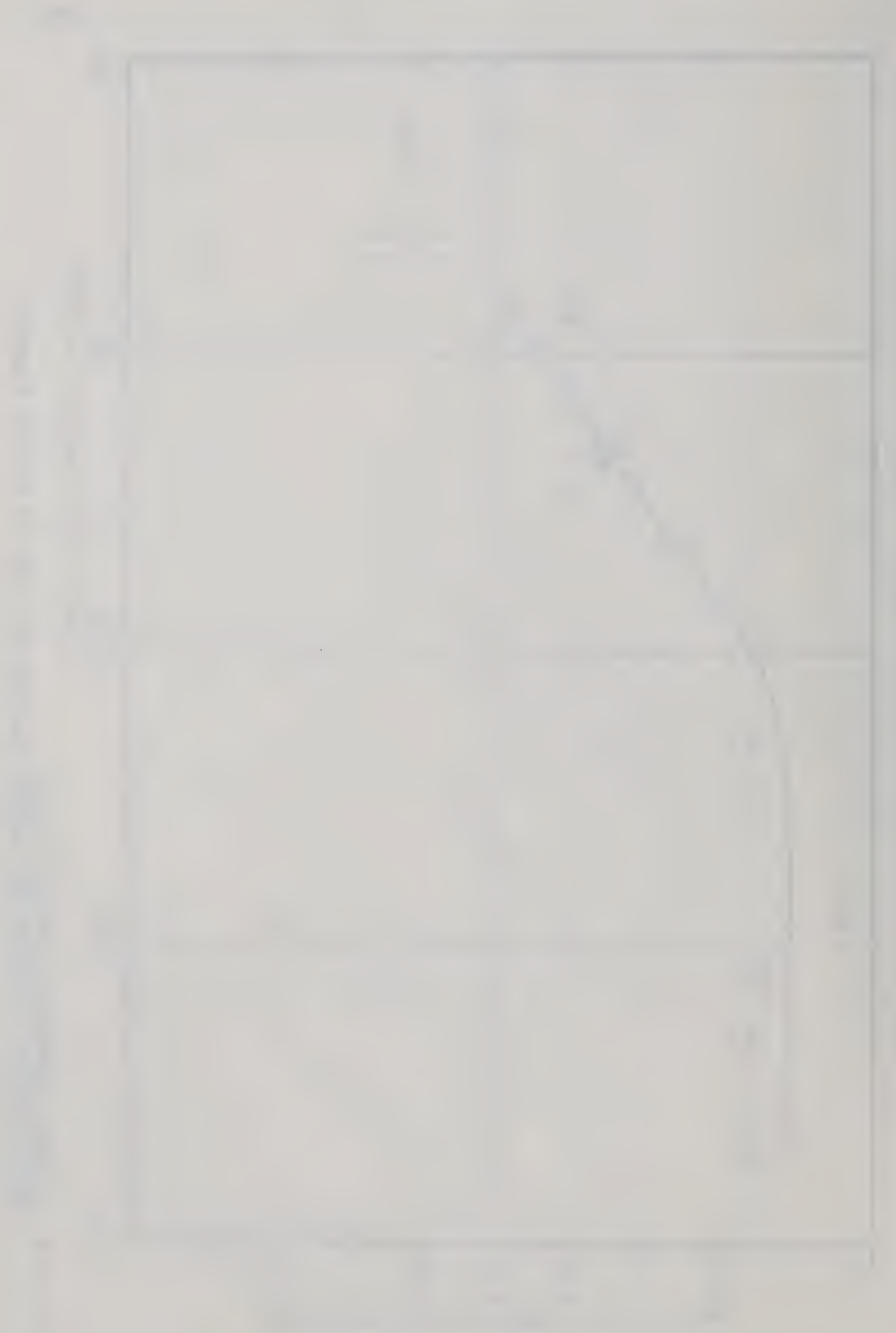


FIG. 3.4. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED FULL STRENGTH LOSS VERSUS TIME FOR 50% LOSS OF INITIAL STRENGTH.



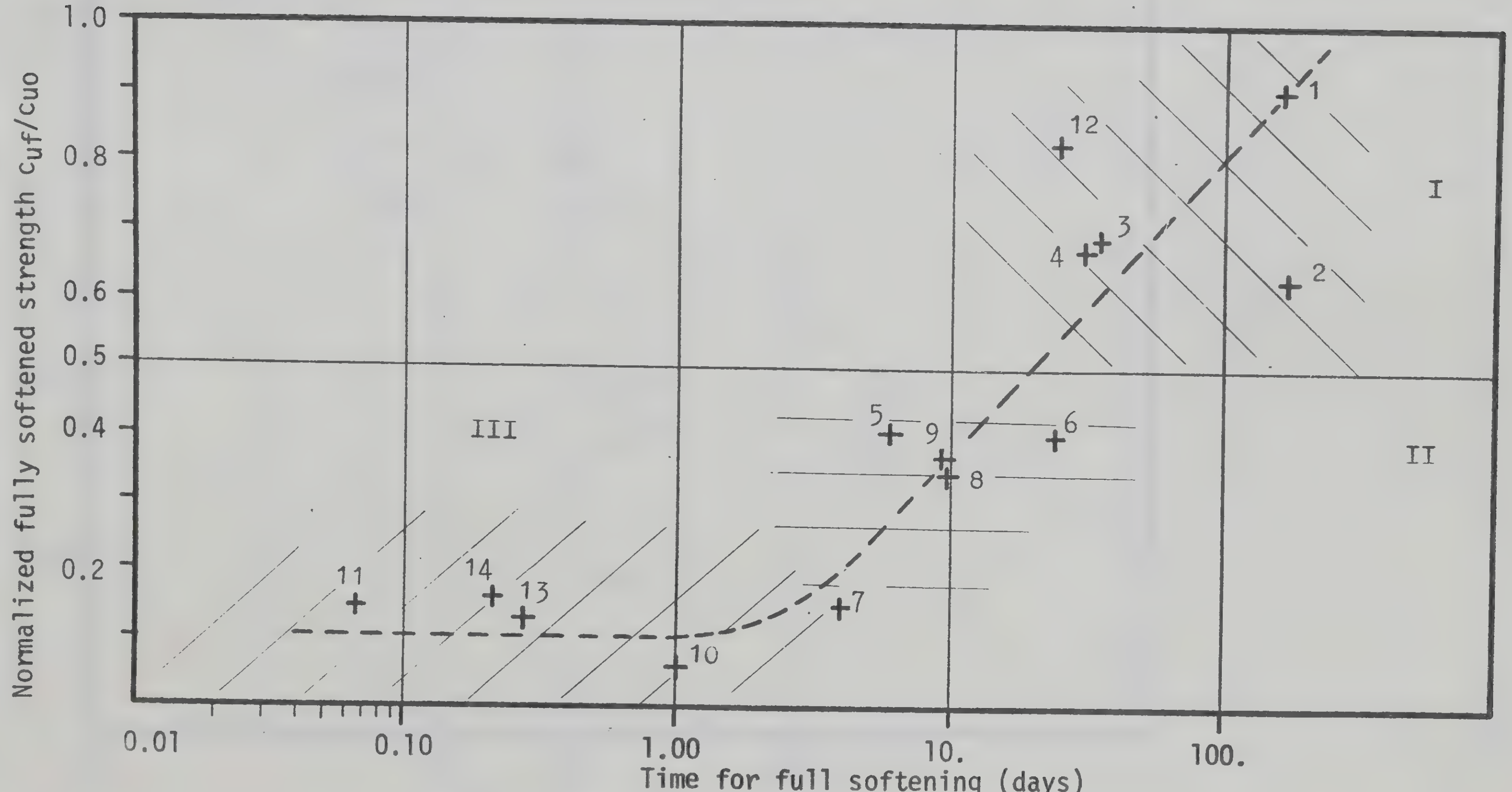


FIG. 3.5. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED FULLY SOFTENED STRENGTH VERSUS TIME FOR FULL SOFTENING.

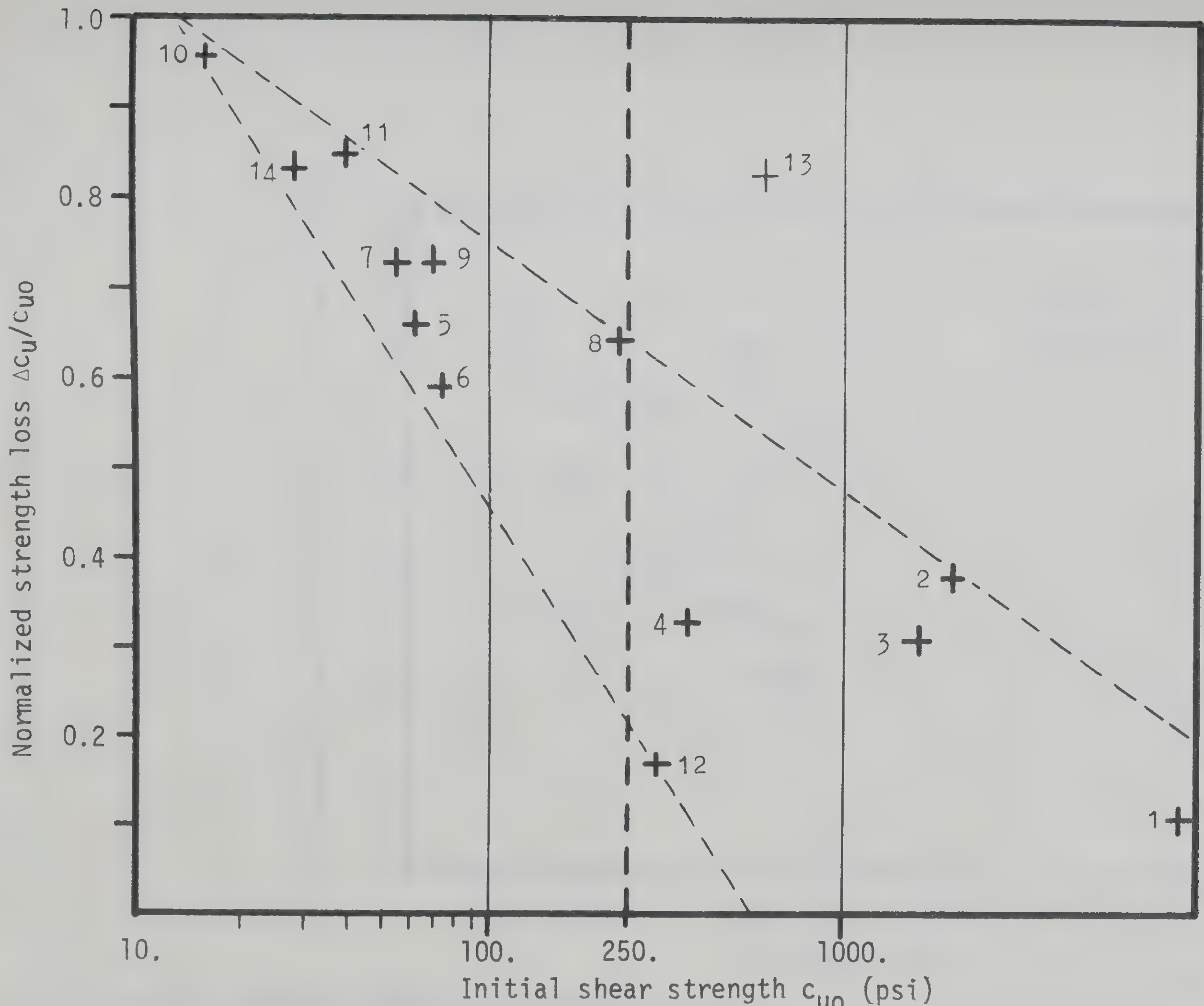


FIG. 3.6. STANDARD COMPRESSION SOFTENING TEST: NORMALIZED FULL STRENGTH LOSS VERSUS INITIAL SHEAR STRENGTH

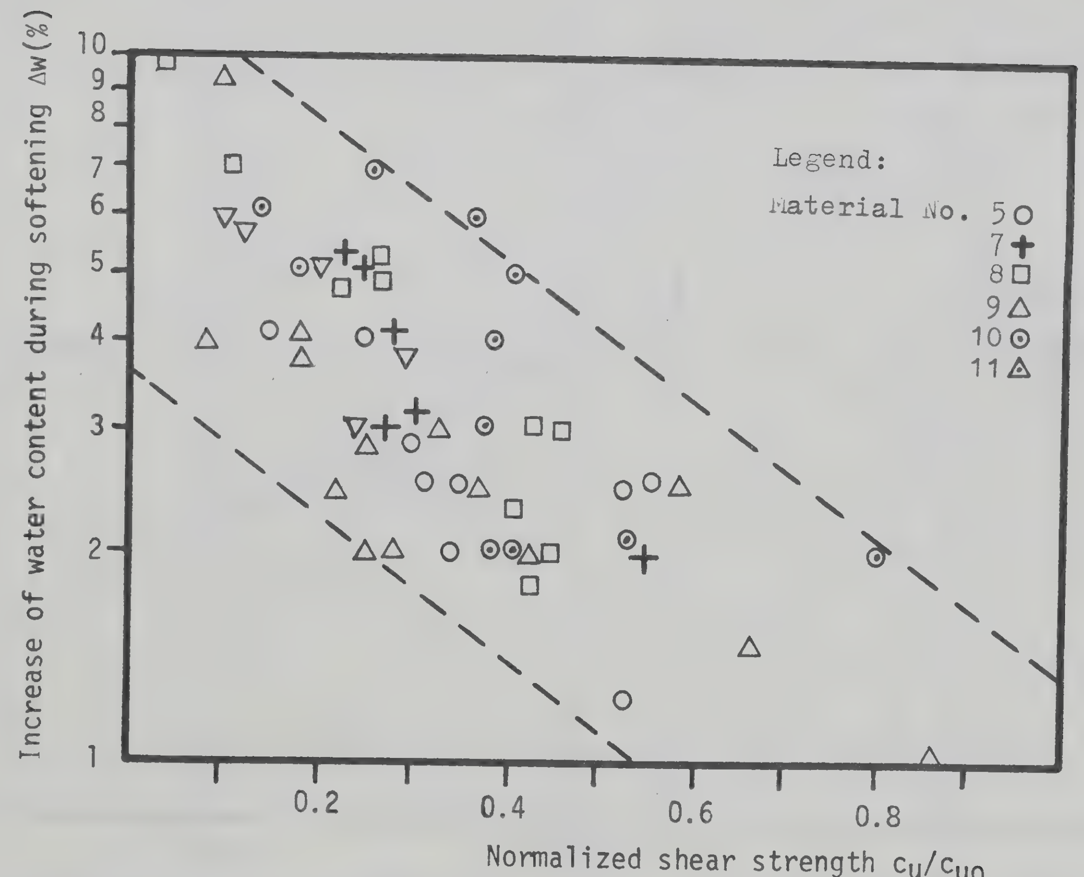


FIG. 3.7. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED SHEAR STRENGTH VERSUS CHANGE OF WATER CONTENT FOR ALL MATERIALS.

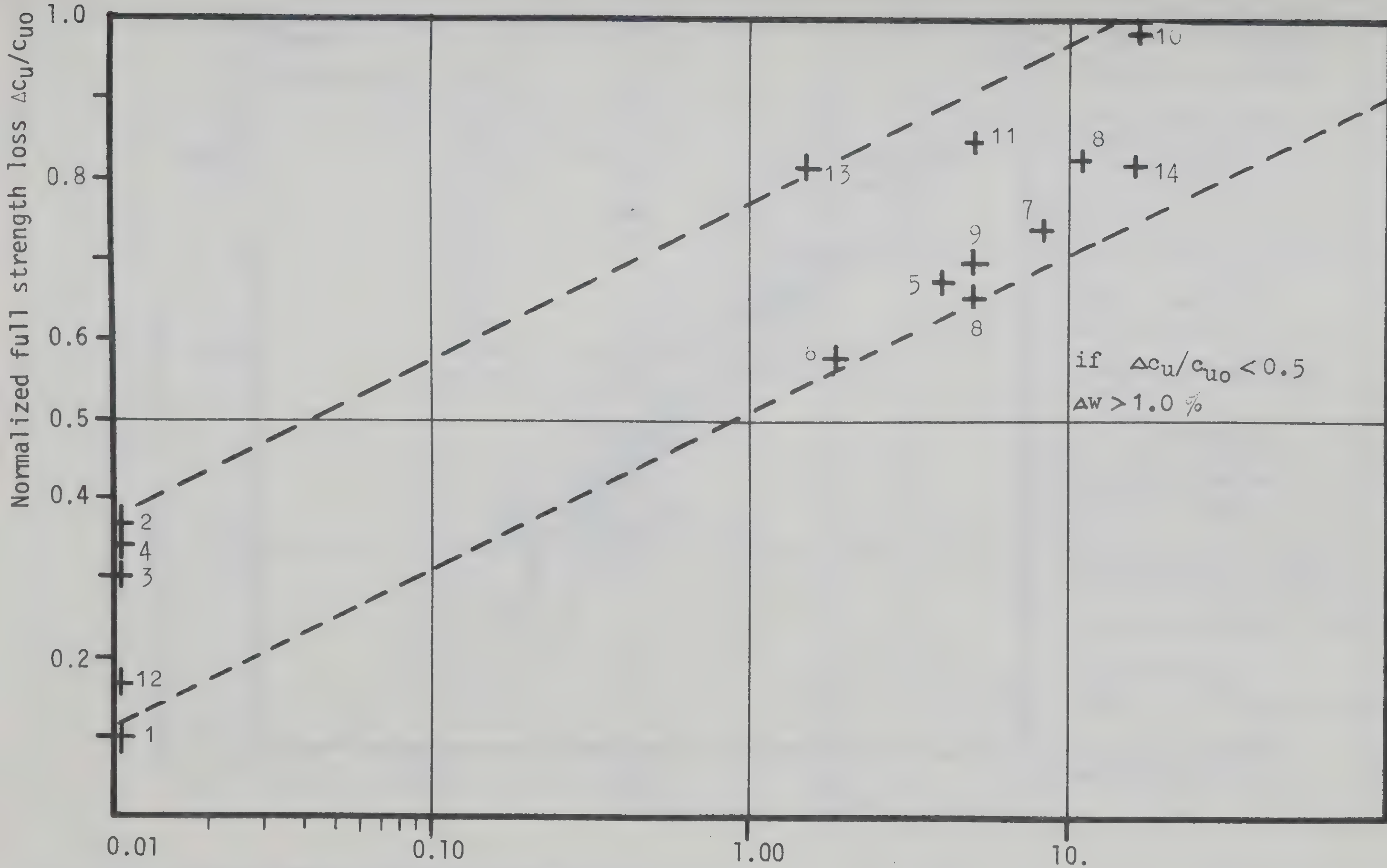


FIG. 3.8. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED FULL STRENGTH LOSS VERSUS MAXIMUM CHANGE OF WATER CONTENT.

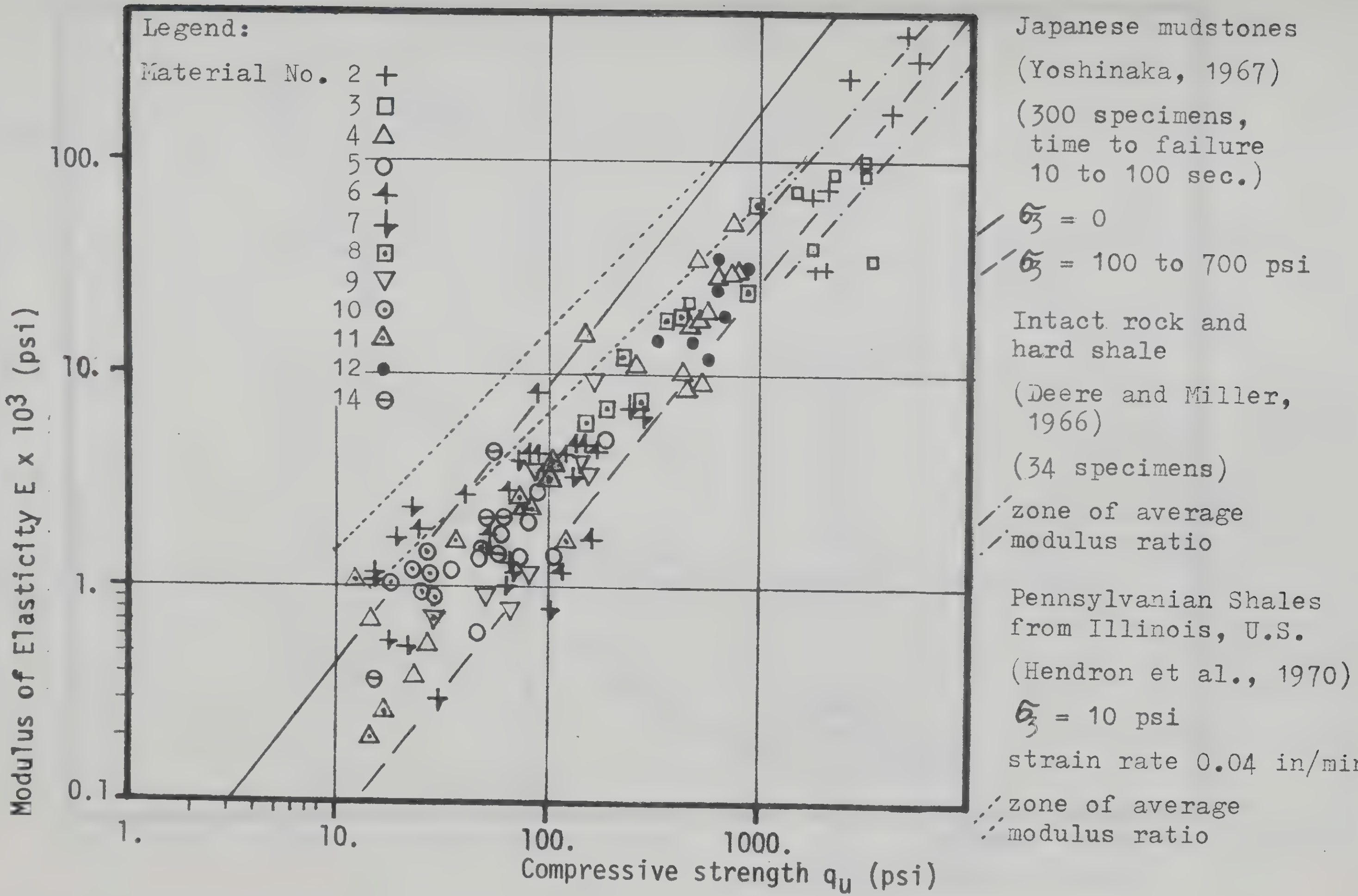


FIG. 3.9. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED MODULUS OF ELASTICITY VERSUS COMPRESSIVE STRENGTH FOR SOFTENED AND UNSOFTENED MATERIALS (CELL PRESSURE σ_3= 50 PSI. TIME TO FAILURE: 5 TO 10 MINUTES).

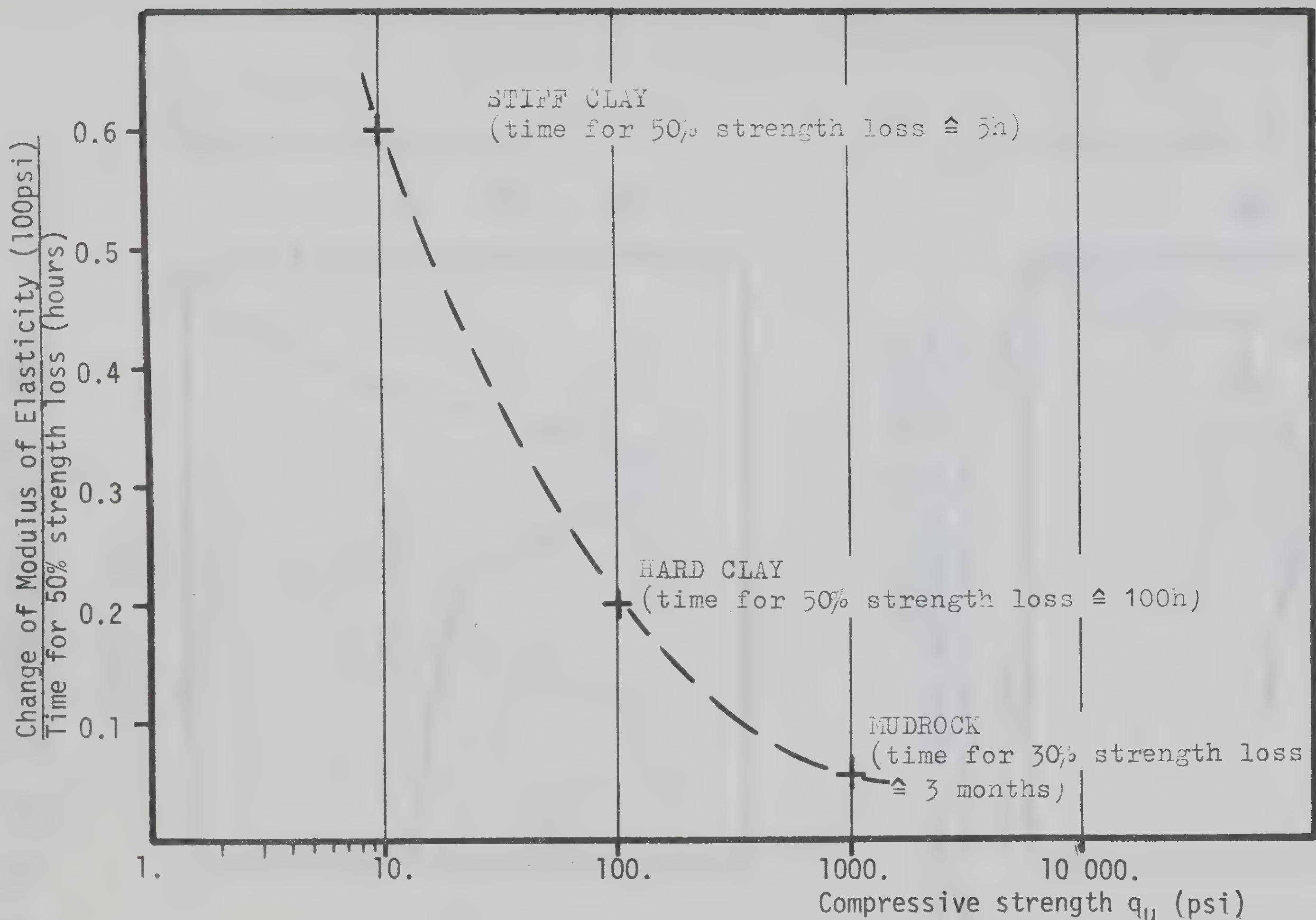


FIG. 3.10. STANDARD COMPRESSION SOFTENING TEST:
AVERAGE CHANGE OF UNDRAINED MODULUS OF ELASTICITY DURING TIME FOR 50% STRENGTH LOSS (30% STRENGTH LOSS FOR MUDROCKS) VERSUS INITIAL COMPRESSIVE STRENGTH.

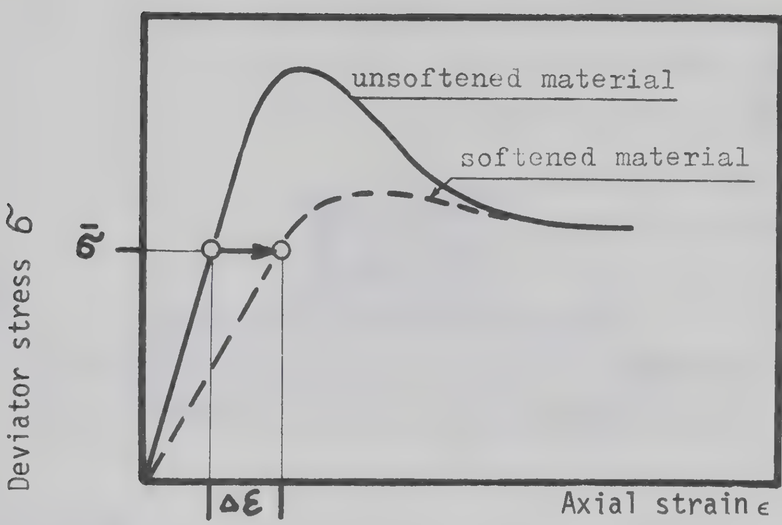


FIG. 3.12. STRESS STRAIN CURVES OF SOFTENED AND UNSOFTENED MATERIAL TO DEMONSTRATE STRAINS CAUSED BY DECREASE OF STIFFNESS.

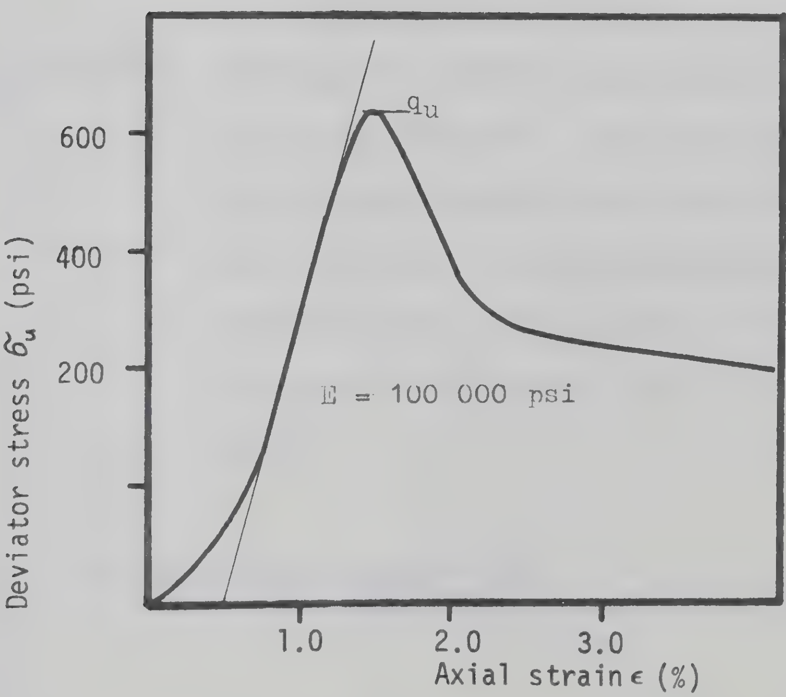
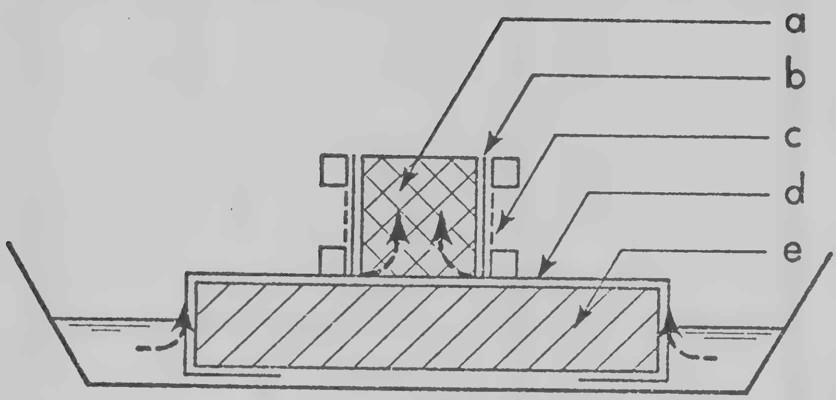


FIG. 3.11. STANDARD COMPRESSION SOFTENING TEST: STRESS STRAIN CURVE INDICATING THAT THE MODULUS OF ELASTICITY WAS SELECTED AS THE SLOPE OF THE LINEAR PORTION OF THE CURVE.



a) SPECIMEN

b) FILTERPAPER between specimen and container walls

c) BRASS CONTAINER with perforated walls

d) WET FILTERPAPER which supplies the water which is absorbed into the sample.

e) GLASSPLATE sitting in a container which was filled with distilled water. The filterpaper will be kept constantly wet during the wetting process by being wrapped around this glassplate; the specimens were placed on top of the glassplate and the filter-paper.

FIG. 3.13. QUANTITATIVE SLAKING TEST:
THE WETTING STAGE (TEMP. T = 68^{0}F, 90% HUMIDITY).

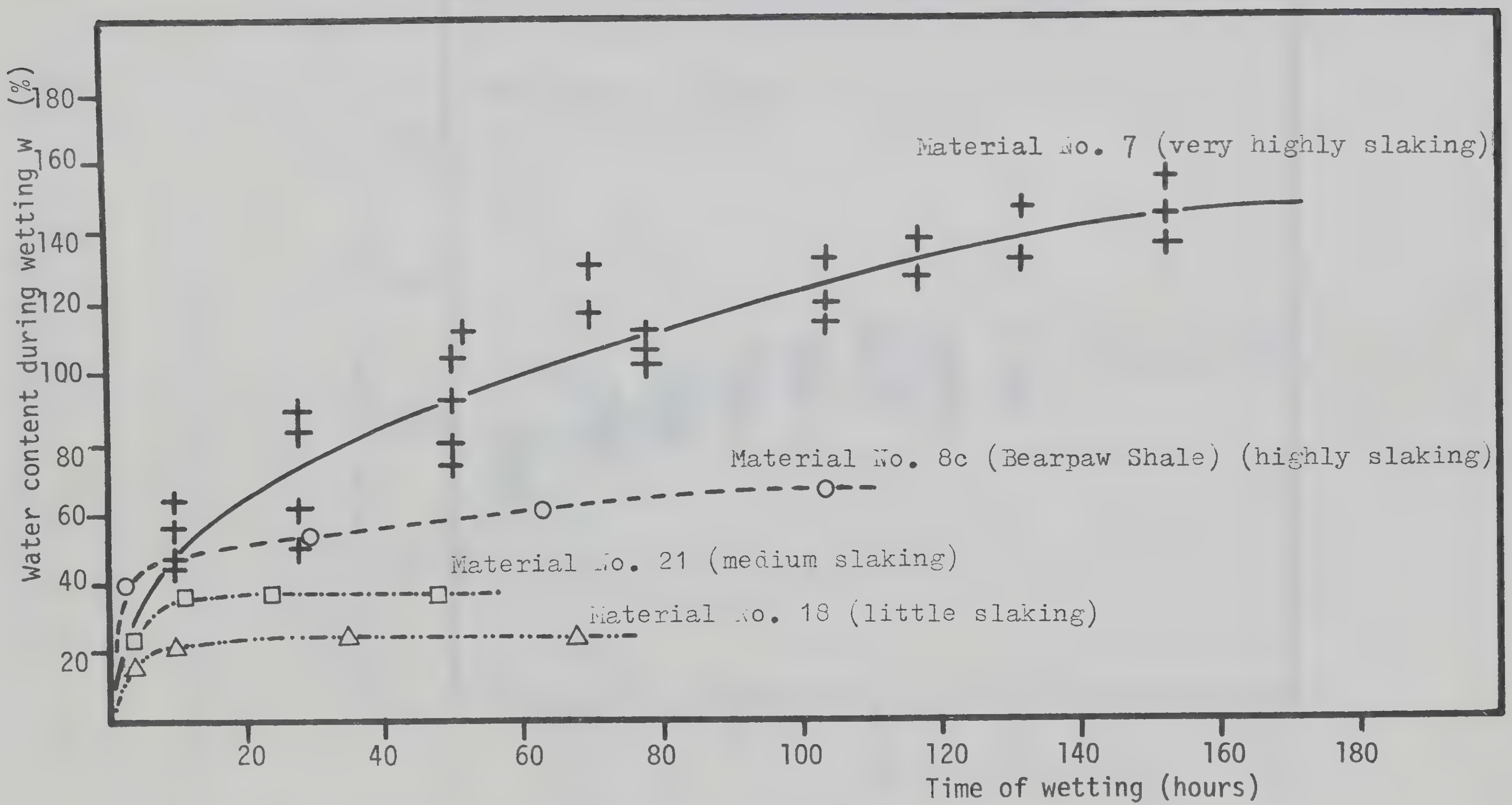
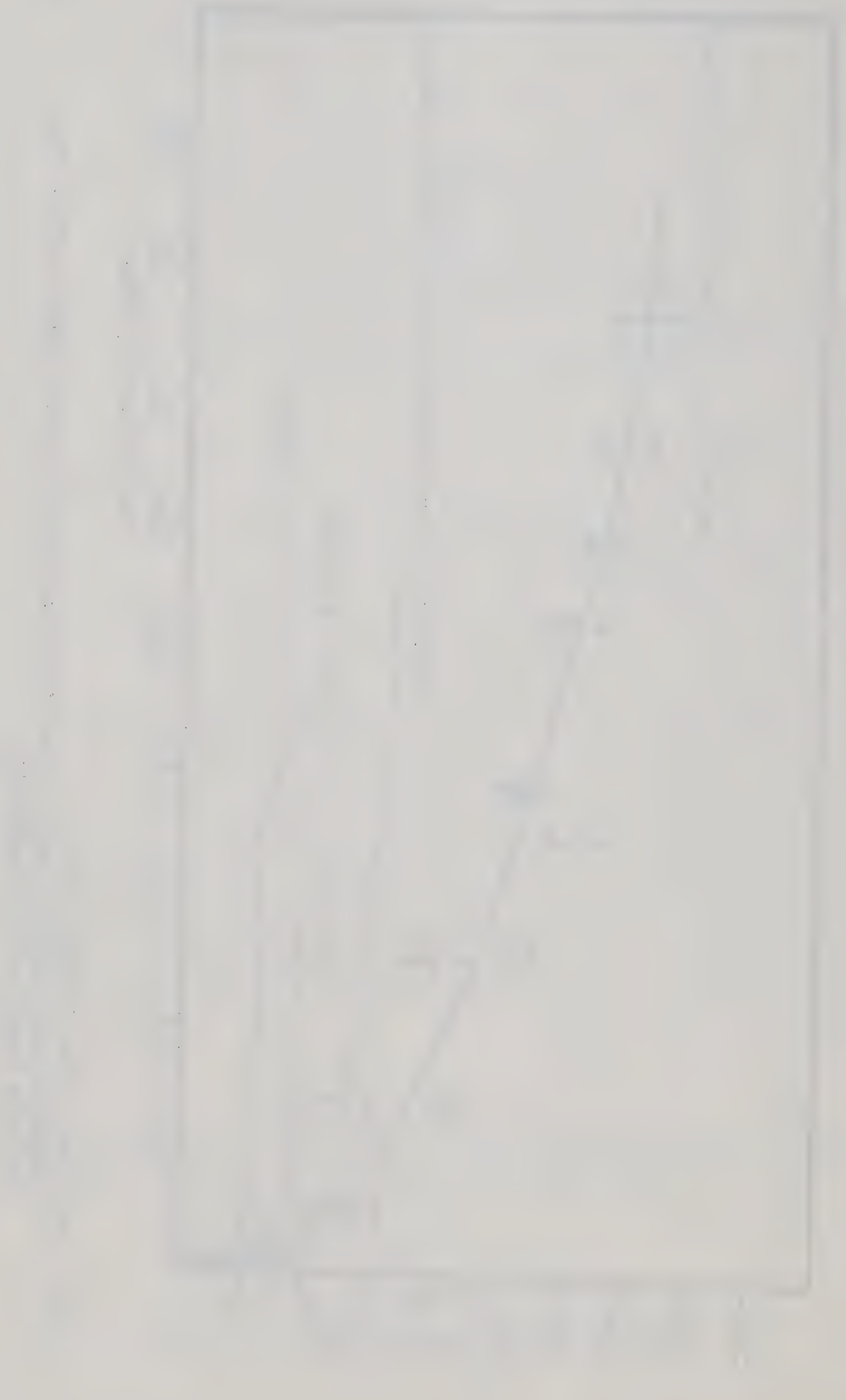


FIG. 3.14. QUANTITATIVE SLAKING TEST:
CHANGE OF WATER CONTENT VERSUS TIME OF WETTING DURING THE VERY FIRST WETTING STAGE FOR FOUR REPRESENTATIVE MATERIALS.



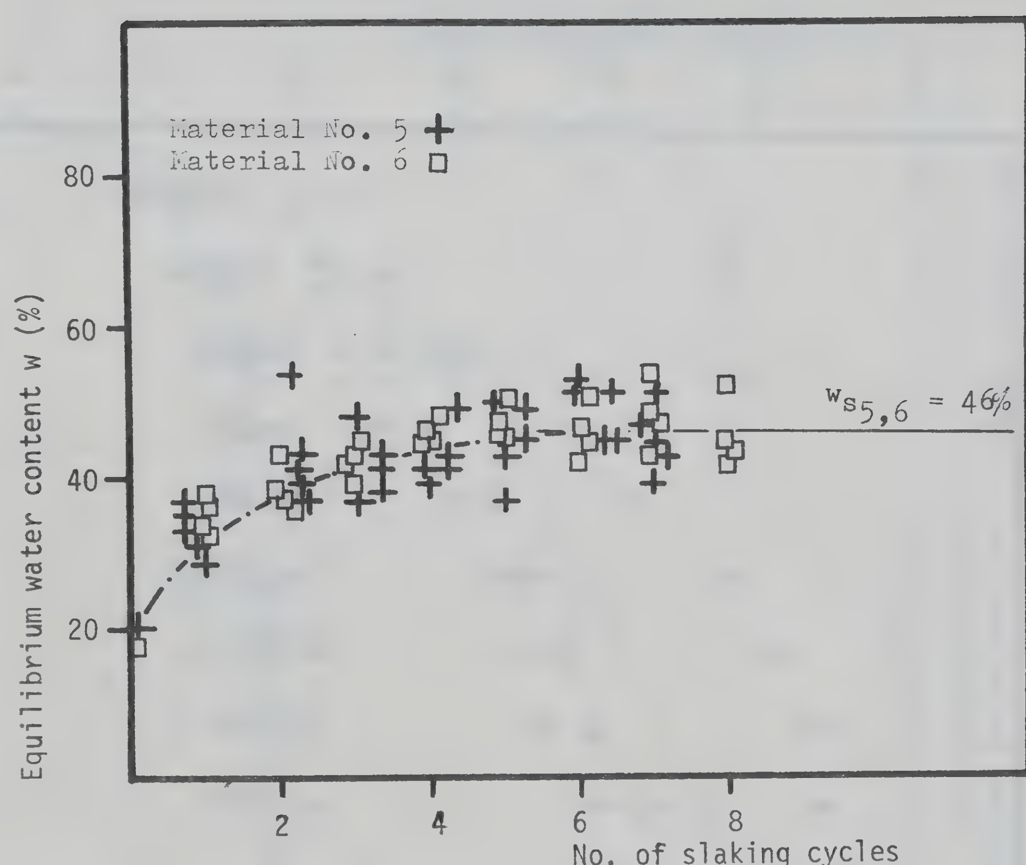


FIG. 3.15. QUANTITATIVE SLAKING TEST:
CHANGE OF WATER CONTENT WITH NO. OF SLAKING CYCLES
FOR MATERIALS NO. 5 AND 6.

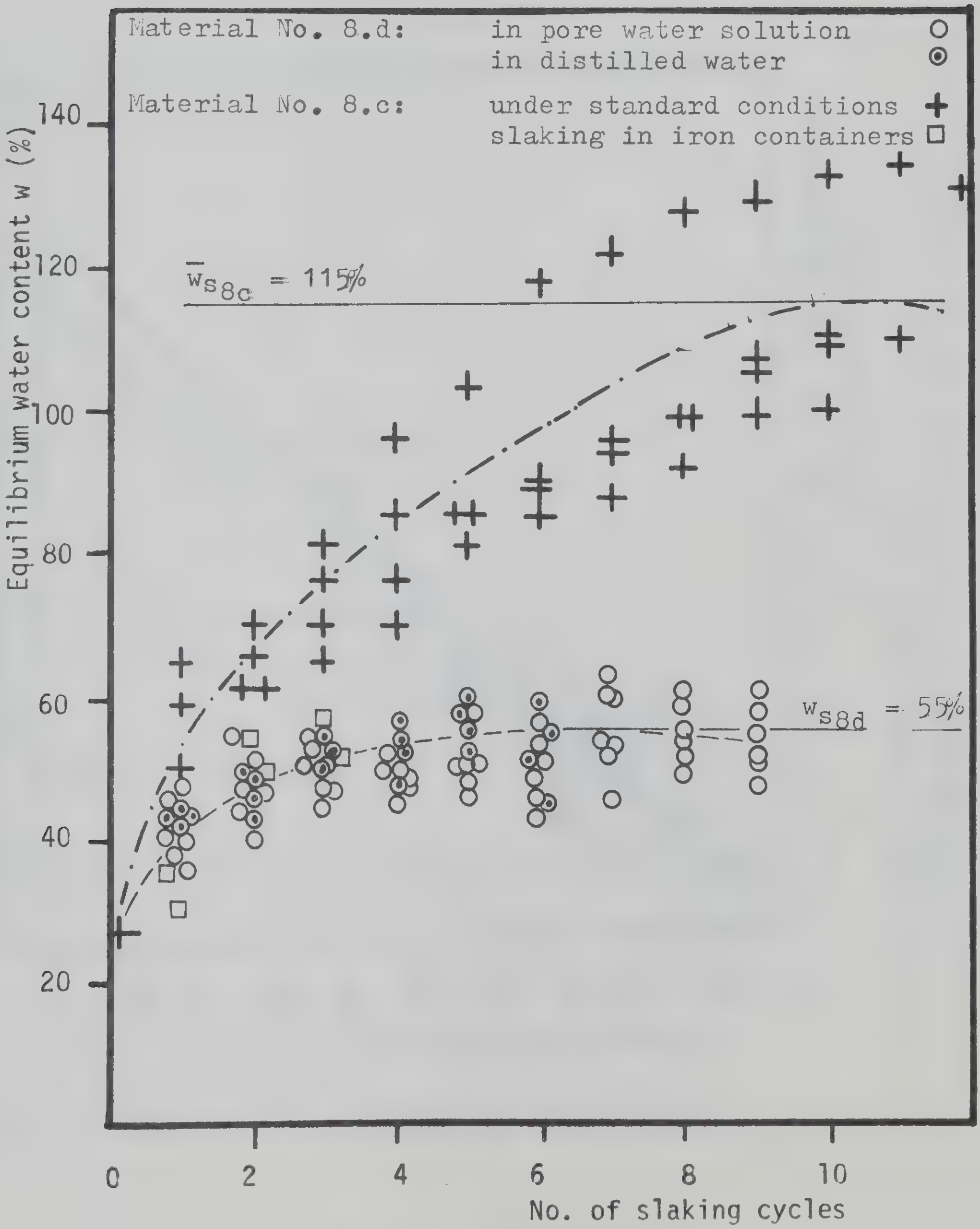


FIG. 3.16. QUANTITATIVE SLAKING TEST:
CHANGE OF WATER CONTENT WITH NO. OF SLAKING CYCLES
FOR MATERIALS NO. 8c AND 8d (BEARPAW SHALE).

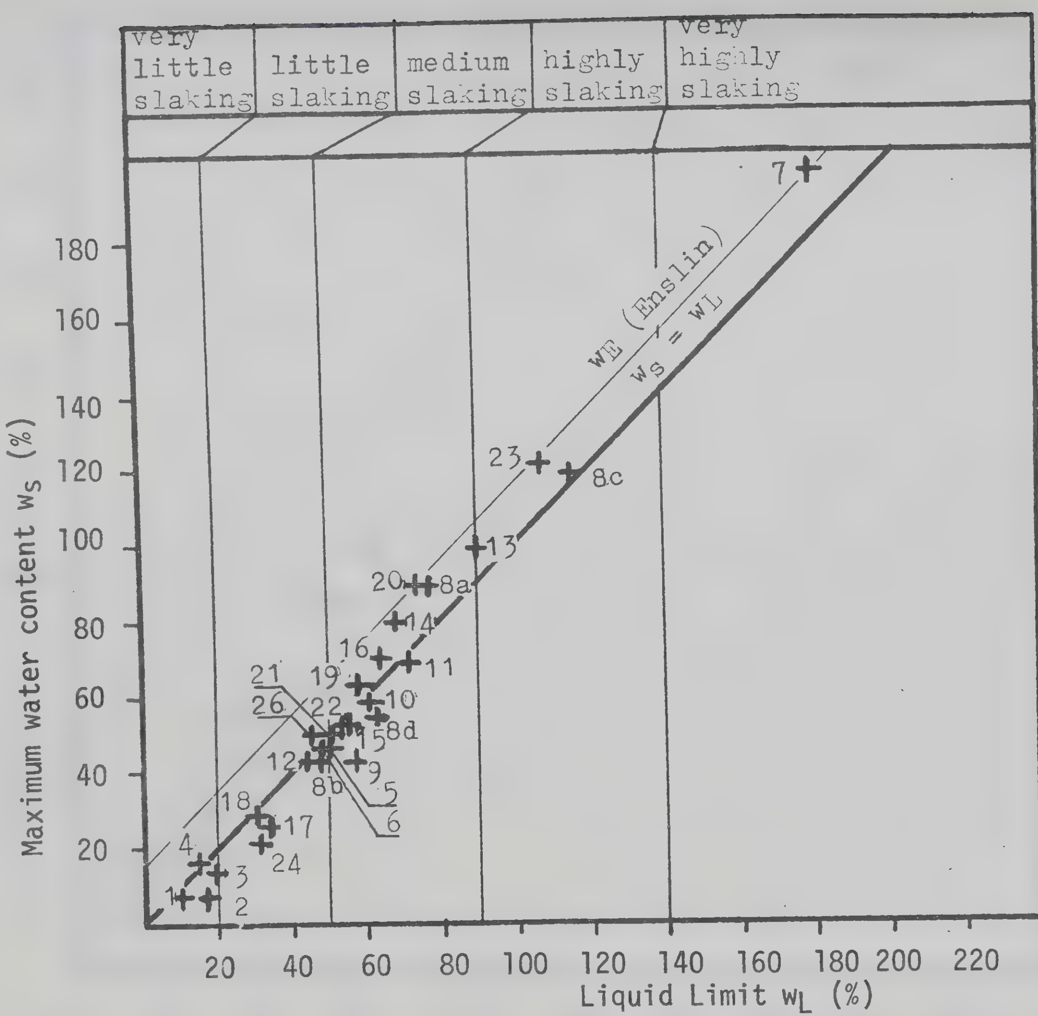


FIG. 3.17. QUANTITATIVE SLAKING TEST:
WATER CONTENT AFTER FULL SLAKING VERSUS LIOUID LIMIT.

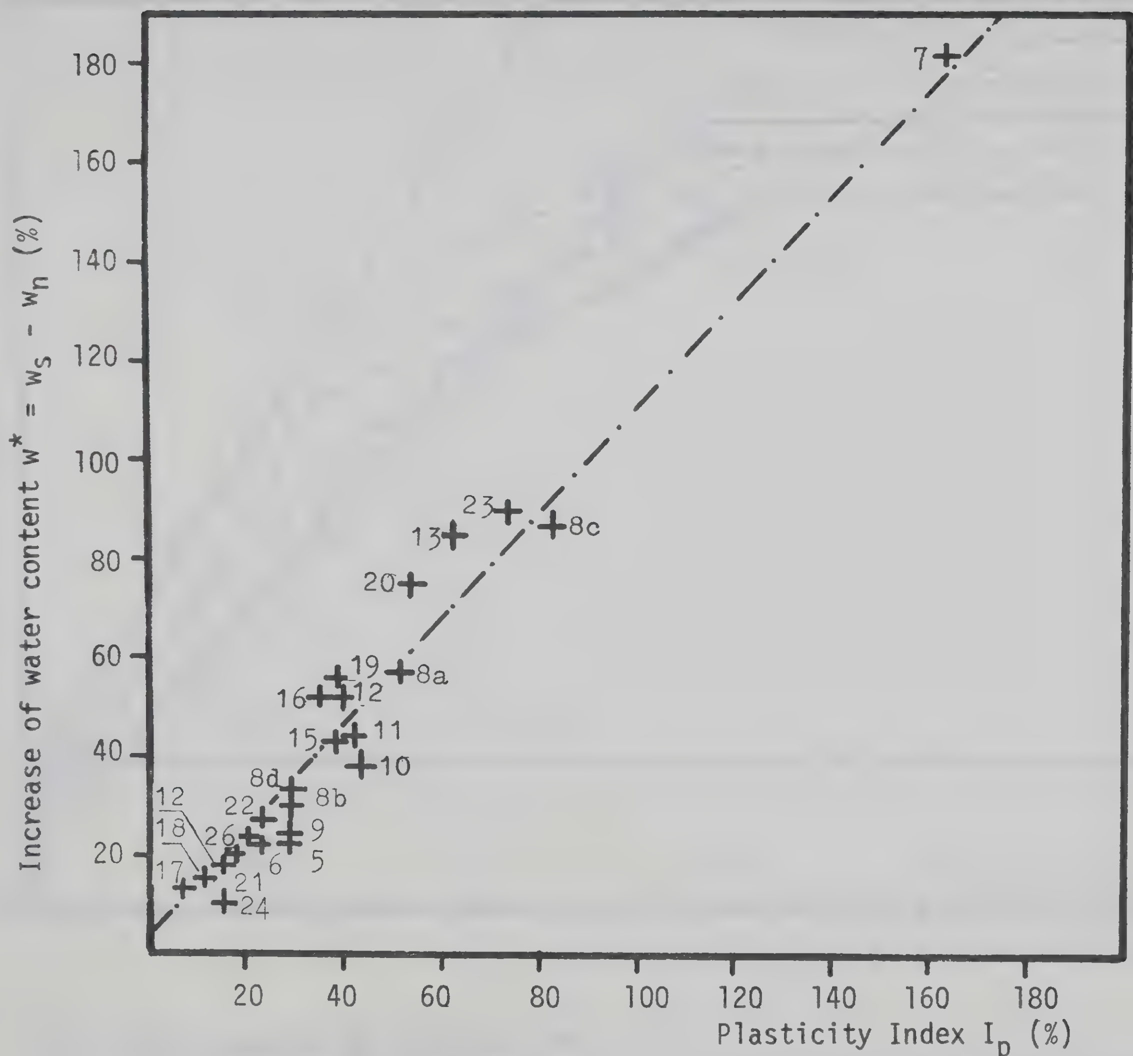


FIG. 3.18. QUANTITATIVE SLAKING TEST:
MAXIMUM INCREASE OF WATER CONTENT VERSUS PLASTICITY INDEX.

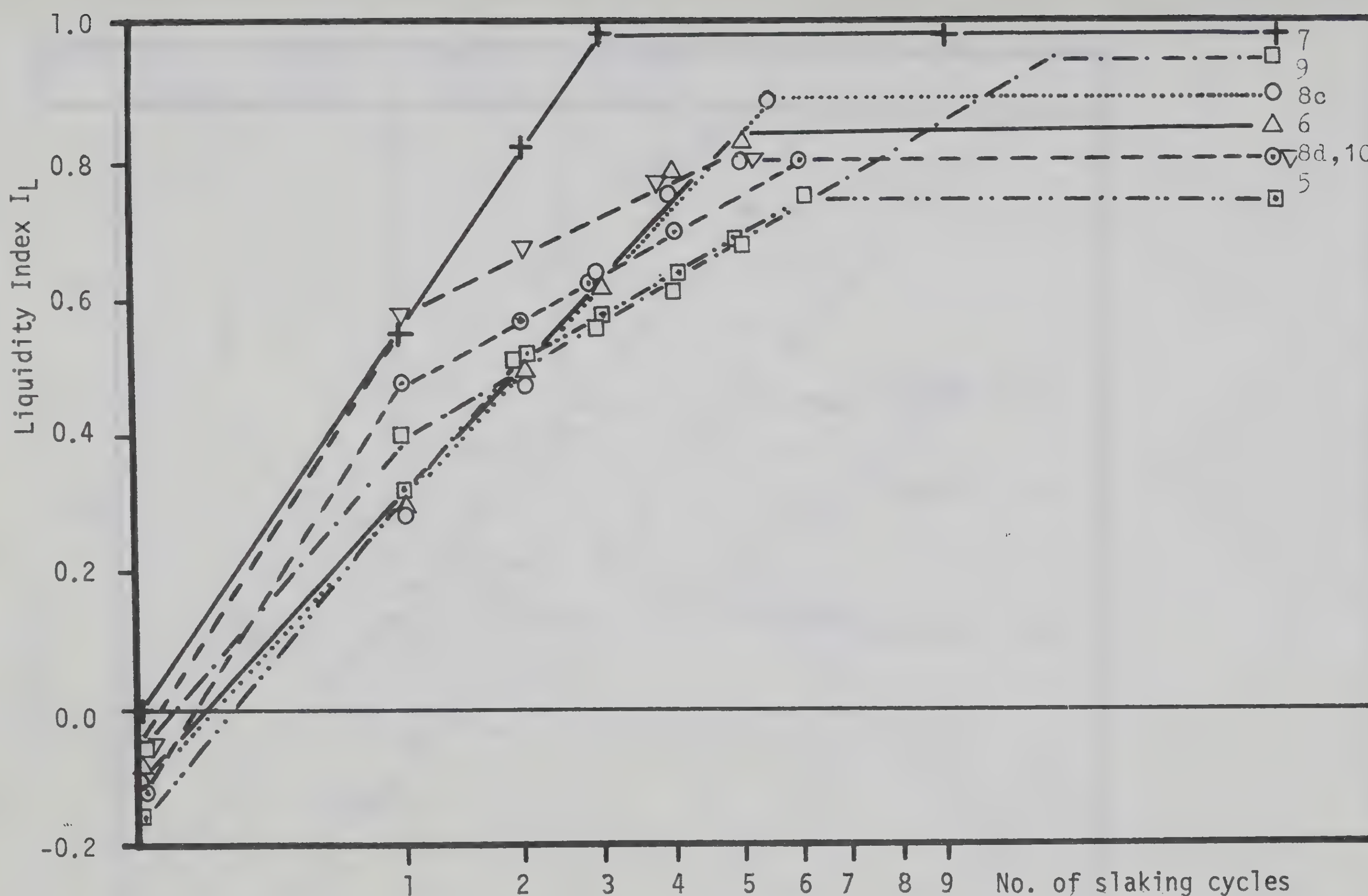


FIG. 3.19. QUANTITATIVE SLAKING TEST:
LIQUIDITY INDEX VERSUS NO. OF SLAKING CYCLES IN A SQUARE ROOT PLOT.

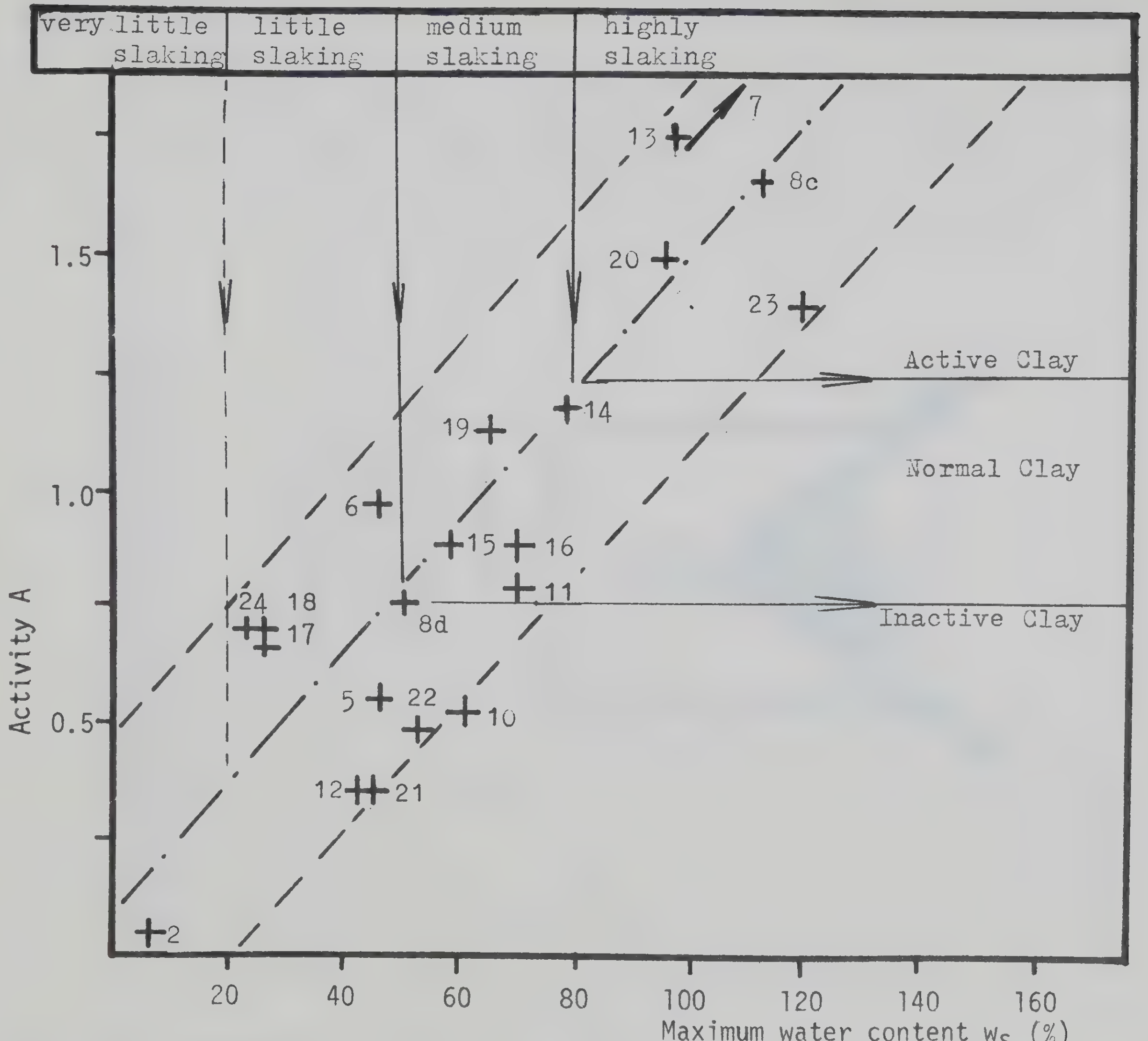
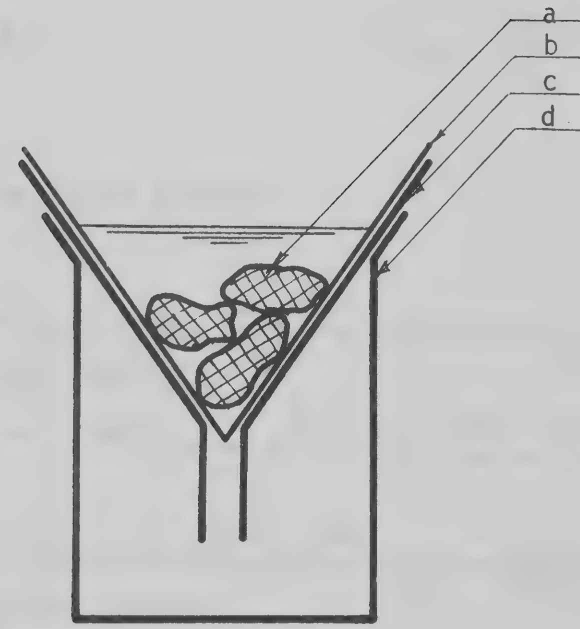


FIG. 3.20. QUANTITATIVE SLAKING TEST: ACTIVITY VERSUS WATER CONTENT AFTER FULL SLAKING.



a) Specimen

b) Filter Paper

c) Glas-Funnel

d) Glas-Container

FIG. 3.21. SET-UP FOR RATE OF SLAKING TEST.

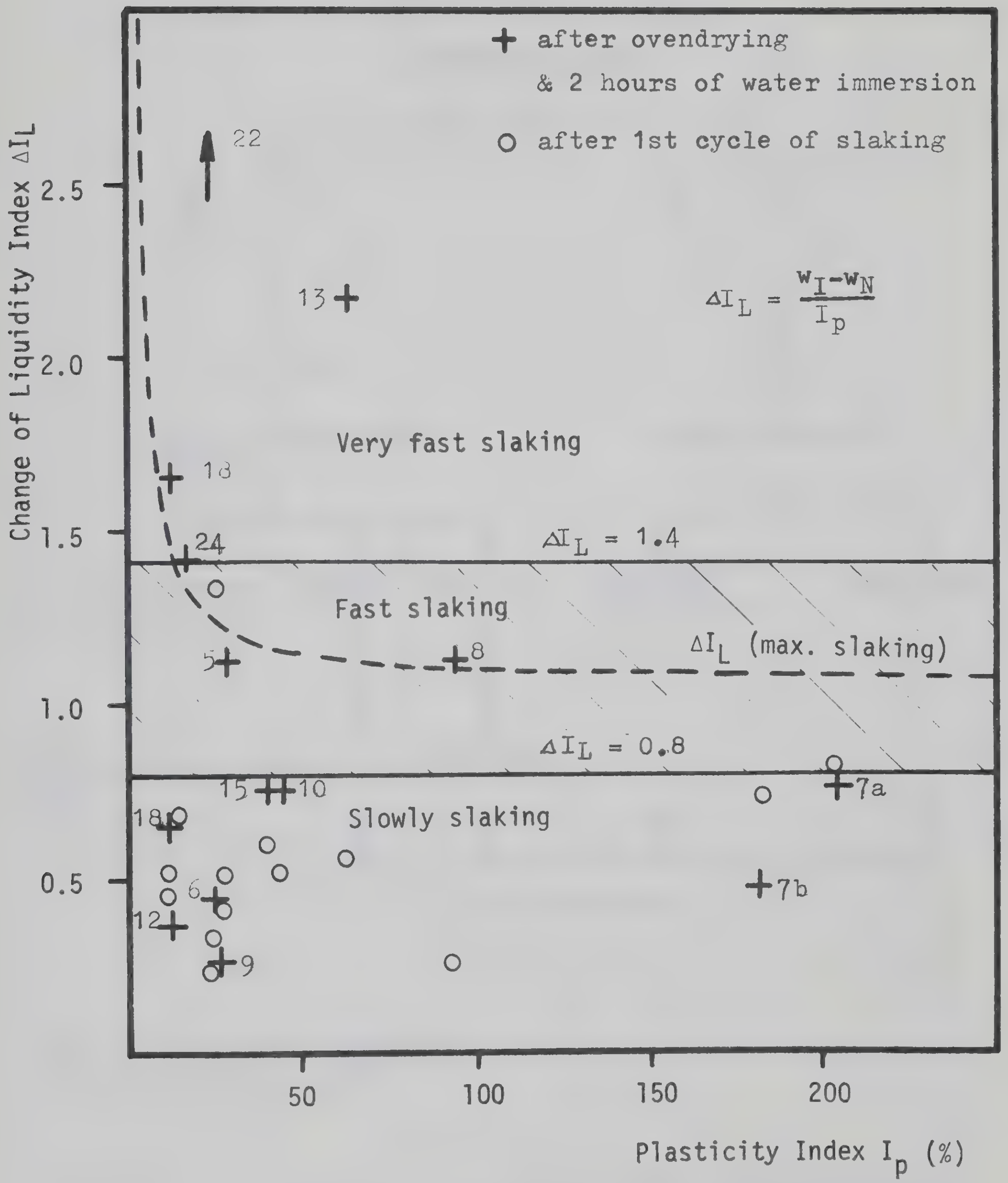
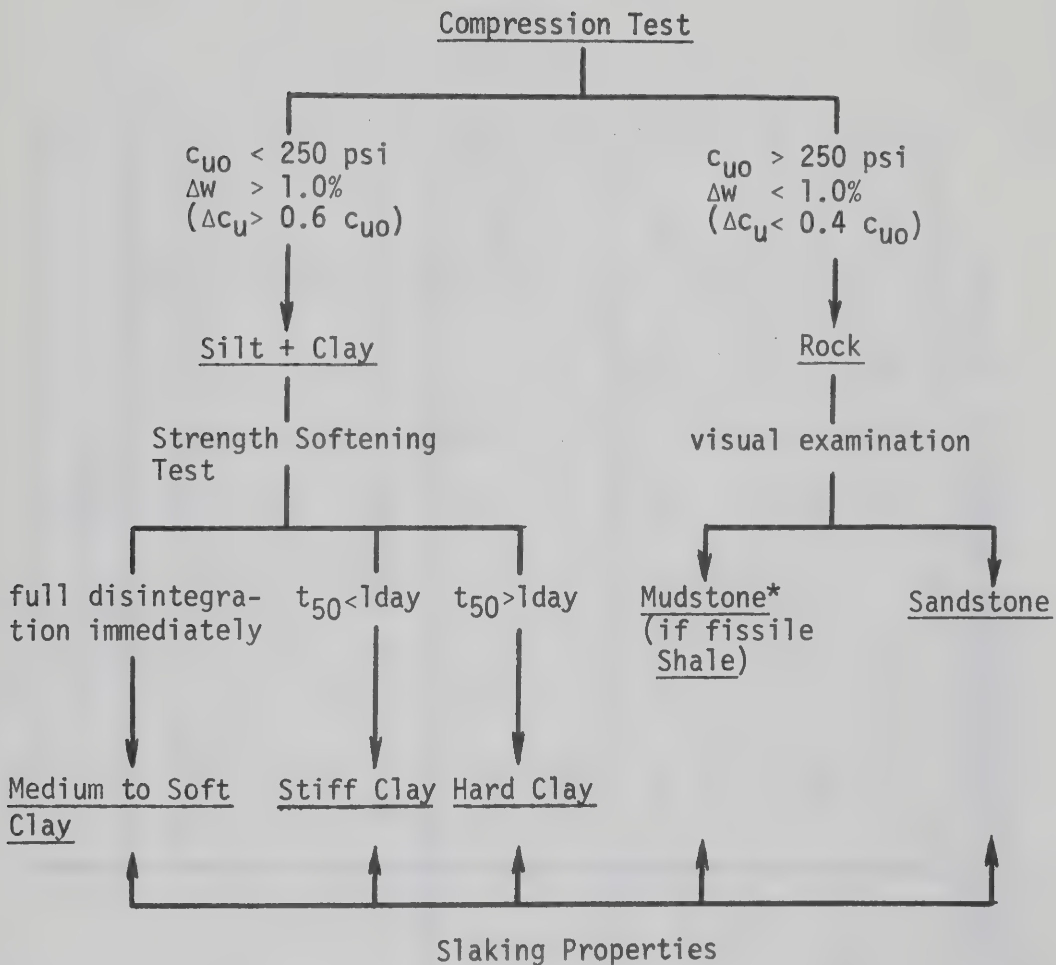


FIG. 3.22. RATE OF SLAKING TEST:
CHANGE OF LIQUIDITY INDEX VERSUS PLASTICITY INDEX.



* The terms rock and stone may be used alternatively.

FIG. 3.23. CLASSIFICATION SYSTEM FOR INORGANIC, NONCALCAREOUS, SEDIMENTARY MATERIALS ON THE BASIS OF THE STANDARD COMPRESSION SOFTENING TEST RESULTS.

Rate of Slaking: $\Delta I_{L1} = \frac{I_{L1} - I_{L0}}{\text{2h water immersion}}$		Amount of Slaking: $w_S = w_L$				
		very low VL $w_L < 20$	low L $20 < w_L < 50$	medium M $50 < w_L < 90$	high H $90 < w_L < 140$	very high VH $w_L > 140$
	slow S $\Delta I_L > 1.4$	VS S	L S	M S	H S	VH S
	fast F $0.8 < \Delta I_L < 1.4$	VS F	L F	M F	H F	VH F
	very fast VF $\Delta I_L < 0.8$	VS VF	L VF	M VF	H VF	VH VF

FIG. 3.24. SLAKING PROPERTIES FOR INORGANIC, NONCALCAREOUS, SEDIMENTARY MATERIALS ACCORDING TO THE QUANTITATIVE SLAKING TEST RESULTS AND THE RATE OF SLAKING TEST.

CHAPTER IV

CASE HISTORIES

1. General

In a large number of theoretical considerations (e.g. Taylor, 1948; Skempton, 1964; Bishop, 1967; Conlon (Peck, 1967); Bjerrum, 1967; Duncan and Dunlop, 1968; Christian and Whitman, 1968; Gates et al., 1972) it was shown that mechanisms, such as nonhomogeneous stress distribution, local overstressing, release of horizontal stresses, etc. can possibly create progressive failure in a soil by which cohesion and angle of friction are reduced to their residual values. Critical investigations of case histories showed that mechanisms of progressive failure affecting c' and ϕ' were not apparent in many slopes (e.g. Turnbull and Hvorslev, 1967; James, 1970). It appeared promising to continue in this line and to approach the problem of progressive failure on the basis of field observations, rather than to investigate another theoretical model. Therefore case histories will be analysed which have been reported as examples in which progressive failure reduced the effective angle of friction of a soil, as well as case histories which show contradicting evidence.

Many of the reported case histories are only of limited

value, because most of them lack complete data of the conditions at failure. The most severe deficiency is usually lack of dependable pore pressure measurements. The importance of detailed pore pressure data is illustrated by the case history of the Devon Slide (Eigenbrod and Morgenstern, 1971), in which less detailed information would have yielded a completely different interpretation.

2. First-Time Failures where the Mobilized Average Strength Parameters along the Slip Plane appeared to be $c'=0$ and $\phi' < \phi'_{peak}$

First-time failures in which the average mobilized strength parameters are $c' = 0$ and $\phi' < \phi'_{peak}$ would show that there are mechanisms of progressive failure which cause a reduction of ϕ' as well as of c', in practice.

2.1. Devon Slide (Eigenbrod and Morgenstern, 1971)

The Devon Slide is an example of a slide which, seemingly a first-time slide, mobilized the residual angle of shearing resistance along the slip surface. The Devon landslide is located adjacent to Highway No. 60 where it crosses the valley of the North Saskatchewan River near the townsite of Devon about 12 miles upstream of Edmonton, Alberta. The location is illustrated in Fig. 4.1. During the summer of 1965 highway improvements were undertaken and the road leading down the valley wall was reconstructed to reduce grades and ease horizontal curves. Parts of the pre-existing slopes

on the North wall of the valley were cut back to 2.75:1, i.e. , about 20 degrees to the horizontal. In the vicinity of the slope under consideration, the vertical difference in elevation between the top of the cut and the ditch invert is about 60 feet. Prior to construction in March, 1965, tension cracks had been observed in the slope close to where the scarp of the slide was to be. The landslide occurred in the fall of 1965. A surface examination of the slide was made at that time by Thomson and Broscoe (1968) and subsurface investigations were initiated by Eigenbrod and Morgenstern (1971) in the summer of 1969.

Geology of area

The landslide occurred in the bedrock of the area which is the Upper Cretaceous Edmonton Formation. The Edmonton Formation is a brackish to fresh-water deposit and consists of slightly indurated interbedded mudstone, claystone, siltstone and sandstone. The general dip is southwesterly at 20 feet per mile. Coal seams are present ranging from a few inches to several feet in thickness. Layers of pure bentonite with thicknesses between an inch and about one foot may also be found. The presence of bentonite which is derived from altered volcanic ash seriously affects the engineering characteristics of the strata.

At the close of the Cretaceous period, sedimentary deposition slowly changed from a deltaic environment to terrestrial conditions. The deposits laid down in this manner are referred to as the

Paskapoo Formation. During the Oligocene, regional uplift occurred and subaerial erosion began, removing about 2000 feet of sediments of the Paskapoo and the upper part of the Edmonton Formation. The only recorded advance of ice into the Edmonton Area during the Pleistocene was of Wisconsin age (Bayrock and Hughes, 1962). The till sheets can be differentiated, but from a geotechnical point of view they are similar enough to be considered as one unit. The thickness of the till sheets ranges between 20 and 30 feet. The material is dense but often contains conspicuous vertical joints.

Contorted bedrock due to the ice movement can be seen in the vicinity of the landslide. Indeed, the upper portion of the bedrock at the Devon Slide, immediately below the till, exhibits some ice shove features. Numerous sand dunes have developed on the North side of the river following the retreat of the ice. The river valley was cut into the bedrock relatively quickly during the ice retreat and many landslides developed along the valley walls. These slides are still characteristic of the topography of the valley slopes.

Field investigations

Five pits were excavated into the toe of the slide in order to locate the slip plane and to obtain block samples of sheared material for subsequent testing. A detailed description of Pit No. 3 is given in Fig. 4.3. The location of the pits is shown in Fig. 4.2.

The slip surface was found in the pits. The slide had occurred in a soft greenish white, granular bentonitic clay layer

with a thickness varying from less than one inch to more than one foot. The bentonite was underlain by coal and the main slip surface could be clearly identified in the clay at the bottom contact with the coal. Secondary shear planes could sometimes be seen within the bentonite about one inch above the main shear plane and they were occasionally iron stained. Block samples containing the bentonitic layer were cut and transported back to the laboratory for further testing. Thin-walled sampling tubes, 1 1/2 inches in diameter, were pushed by hand across the failure surface to obtain samples for triaxial testing of the sheared material. About 15 hours after excavation, free water which had seeped in mainly from the coal could be observed in Pits Nos. 4 and 5. The upper surface of the bentonite was irregular, having a surface characteristic of deformation during deposition. Broken, abundantly iron stained claystone was found at the upper contact and it was wet in places. This zone was not identified in borehole samples.

The coal layer that was found in the pits outcropped on the East flank of the slide where it was bent upwards for about 1.5 feet. On the margin of the slide, where the contortion of the coal started, a small spring was observed that discharged water throughout the year. Other thin coal layers which appeared strongly contorted were observed halfway up the slope.

Six boreholes were drilled in the back slope of the slide in 1969. Four inch diameter augers were used within the overlying

till deposits and a Pitcher Sampler (Terzaghi and Peck, 1967) was used to recover four inch diameter undisturbed samples of the bedrock from Holes 1, 2, 3 and 4. The location of all holes is shown in Fig. 4.2. Borehole Nos. 5 and 6 were drilled only with augers and therefore no undisturbed samples were recovered from them. For most of the drilling with the Pitcher Sampler, bentonitic mud had to be used to seal the holes and prevent the loss of the circulating water.

The bedrock consists of a variation of materials, containing grey, medium hard brittle mudstone, light grey medium hard to hard bentonitic sandstone, light to dark brown medium soft carbonaceous mudstone, broken coal, and two main layers of greenish white soft bentonitic clay, each about 1 to 4 inches thick. The stratigraphy is given in Fig. 4.4. which is a cross-section through the slide zone. The height of the slope is about 60 feet while the height of the slide is approximately 40 feet. This section also illustrates the amount of excavation that preceded the movements. In the slide zone beneath the scarp, about 15 to 20 feet of till overly a thin coal layer followed by carbonaceous claystone, bentonite, fissured siltstone and claystone, and bentonitic sandstone. This is underlain by the troublesome bentonite layer followed by carbonaceous clay stone and broken coal, varying in thickness from 5 inches to 3 feet. The extreme nonhomogeneity due to material variation and degrees of fissuring is noteworthy. The dark grey mudstone was

often brecciated along the interface with the bentonitic sandstone. Softened material was often found when the spacing between mudstone and sandstone bands was small. The upper bentonite layer was about 25 feet above the one containing the major slip surface. In both clay layers distinct, often dark, failure planes could be observed. It should be emphasized that all boreholes are outside the failure zone yet sheared clay was found in the samples taken from them.

Piezometers were installed in Borehole Nos. 1, 2, 3, 4 and 5 in order to determine the water pressure distribution in the area adjacent to the slip and hence infer the water pressure conditions at the time of the slip. Since the bedrock consists mainly of impermeable bentonitic mudstone and sandstone, the University of Alberta piezometer was used in order to minimize time lag of piezometric response (Brooker, Scott, and Ali, 1968). In Borehole Nos. 1, 2, 4 and 5 the water pressure was determined in the coal directly beneath the bentonite layer at the base of the slide. The water pressure was measured in Borehole No. 3 about 20 feet above the failure surface within a zone that included mudstone and bentonitic sandstone as well as the upper bentonite layer and an additional apparently discontinuous bentonitic clay zone. Continuous readings were taken in Borehole Nos. 1, 2 and 4 over a period of two years and in Borehole No. 3 for one year. The piezometer in Borehole No. 5 was destroyed two weeks after installation. The piezometric observations are given in Fig. 4.5.

The piezometric level in the lower coal is low and is generally slightly below the overlying bentonitic clay layer. The piezometers responded almost instantaneously indicating the free--draining properties of the broken coal. However, the piezometer in Borehole No. 3 revealed a piezometric level near the upper coal layer, about 25 feet above the lower piezometric level. Moreover, a water well belonging to a farmhouse near the site of the slide had been drilled to a depth just above the bedrock and the free water table was reported to vary between a depth of 7 and 13 feet below the ground surface which places it about 20 feet above the second water table. Hence a series of at least three piezometric levels have been detected due to the presence of at least two perched water tables. The distribution is illustrated in Fig. 4.6. This configuration is related to the stratigraphical sequence of impermeable beds separated by very permeable broken coal layers which act as drains. There is a possibility that surface freezing may alter the exit conditions of the groundwater and result in a build-up of water pressure during the winter. Therefore it was of interest to study the seasonal variation in piezometric levels. As may be seen from Fig. 4.5., the variation in the lower level was slight. It rose less than 1 foot in April, shortly after the snow melted. The upper piezometric surface was also quite stable, with an increase in head of only 2 feet recorded in the spring.

After the investigation of 1969 had revealed sheared ben-

tonite behind the slide, it became of interest to determine how far this extended. It also became clear that the Devon Slide, while a first-time slide, was actually located in a massive older slide which had presumably developed during valley formation. Therefore, in the fall of 1970, a seventh borehole was drilled on the top of the valley wall in the area unaffected by valley slumping. As shown in Fig. 4.4., a bentonitic clay layer was detected about 58 feet below the ground surface. This corresponded with the upper bentonite found earlier. The bentonite found in Borehole No. 7 was very hard and did not show any indication of failure. Unfortunately, a lower bentonite layer could not be found because of sample loss just above a coal bed which was thought to correspond with the lower coal layer. A very soft thin zone was found about 15 feet above the lower coal at the interface between light grey sandstone and brownish mudstone. In general, the bedrock material found in Borehole No. 7 appeared harder, more intact, and less iron stained than that found lower in the valley.

Laboratory tests

Three drained triaxial compression tests were performed on 1.4 inches diameter samples obtained by pushing thin-walled tubes obliquely across the failure surface. A horizontally movable load cap was used to inhibit lateral loads from acting on the top of the sample and a rotating pushing to minimize the ram friction. All samples were tested under an effective confining pressure of 30 psi.

Assuming zero cohesion, the residual angle of shearing resistance was found to vary between 5° and 10°. The results of these tests are summarized in Table IV.1. The average residual angle of shearing resistance of the bentonitic clay is 8°.

Sheared specimens of bentonitic clay from block samples obtained in the pits were tested in shear boxes but without satisfactory results. This was due to the difficulty in locating the shear plane in such a way that the direct shear test mobilized the pre-existing surface. This difficulty was particularly severe since the bottom half of the sample was broken coal or carbonaceous mudstone.

The strength of the back slope material is also needed for the stability analysis. As noted earlier, this material varies in composition. It also varies in consistency with harder, dryer, material being subordinate to more broken, softer material. It is possible to distinguish between the softened and unsoftened material and tests have been carried out on both.

The strength of the back slope material has been obtained by conducting consolidated, drained triaxial compression tests on samples of core, 4 inches in diameter. Pore pressure measurements were also made at one end of the sample to ensure that testing was proceeding at a rate slow enough that drainage would be essentially complete. Where this was not practical, the pore pressure observations were used in the calculation of the strength parameters in terms of

effective stress. The tests were carried out under a back pressure of 40 psi to obtain full saturation.

Examples of stress strain curves for both softened and unsoftened back slope material are given in Fig. 4.7. The softened material is more ductile than the unsoftened siltstone, exhibiting a greater strain to failure. The results of all the triaxial tests on back slope material are summarized in Table IV.2. and the shear strength data are plotted in Fig. 4.8. The strength envelope for the unsoftened material is substantially higher than that for the softened material. Based only on the two tests that were performed the shear strength parameters in terms of effective stress are: $\phi' = 19^{\circ}$, $c' = 3720$ psf. These parameters are not defined accurately. Particularly, test value No. 7 has to be interpreted with caution. A negative effective allround pressure σ_3' was recorded which certainly cannot be the true overall value for the sample. The pore pressure values measured are probably true only for the base and the top of the sample. Because of the large size of the specimen and the low permeability of the material nonhomogeneous pore pressure distribution must be expected within the sample. When assuming a more likely effective allround pressure of approximately $\sigma_3' = 0$, the effective shear strength parameters for the unsoftened material become $\phi' = 32^{\circ}$ and $c' = 1700$ psf. The strength envelope for the softened material curves at higher pressures. However, over the average effective stress range of concern in the Devon Slide, the strength of this material is well

represented by $\phi' = 33^{\circ}$ and $c' = 190$ psf.

The shear strength and index parameters are summarized in Table IV.3. It should be noted that the Liquid Limit of the bentonitic clay is only 100%. This is less than is usually found in bentonites of marine origin. Calcium and magnesium are likely the dominant absorbed ion in this clay, as opposed to sodium in the Bearpaw Formation. Since most of the back slope material appeared softened and since it is extremely unlikely that the slip surface would cross intact siltstone, the parameters for the softened material were used in the analysis.

Stability analysis

The calculation of the factor of safety of the Devon Slide has been carried out using the general non-circular limit equilibrium analysis described by Morgenstern and Price (1965). Of particular interest is whether the analysis will predict a factor of safety of unity using measured strength parameters and observed water pressures.

The attitude of the slip surface in the bentonitic clay is known from observations in the pits and borings. The inclination of the back scarp of the slide was measured and it ranges between 30° and 35° to the vertical. The point of emergence of the slide on the slope has also been mapped. Therefore only the details of the slip surface where it passes from the inclined portion to the horizontal section are unknown. Preliminary calculations with assumed strength parameters indicated that the slip surface shown in Fig. 4.9. is the

most critical and detailed calculations have been carried out on it. The results are not very sensitive to variations in the shape and position of this slip surface within the constraints imposed by the field observations.

The relation between the residual shear strength of the bentonitic clay and the Factor of Safety has been computed using the strength parameter for the back slope material cited in Table IV.3. This relation is shown in Fig. 4.10. Zero water pressure was taken as acting on the shear plane in the bentonitic clay while the water pressure in the back slope material was computed from the piezometric level observed in Borehole No. 3. It may be noted from Fig. 4.10. that for the average residual angle of shearing resistance, 8^{o}, the Factor of Safety is 1.01. For the whole range of observed values of residual strength, 5^{o} to 10^{o}, the range in the Factor of Safety is 0.82 to 1.13. Using 8^{o} for the bentonitic clay and putting c' = 0 for the back slope material gives a Factor of Safety of 0.83. It would appear that although c' is difficult to determine with accuracy, it must have been acting when large movements began.

Concluding remarks

The Devon Slide is a first-time occurence of a slip movement. However, it is actually a small portion of a pre-existing large slump block that probably moved during valley formation. The Devon Slide was triggered by an excavation at the toe that removed support from over a bentonitic clay layer. The clay layer was

sheared in place prior to the slide and therefore was at its residual strength. Cracks in the slope prior to the excavation were evidence that the slope was only marginally stable and that some reduction in shearing resistance may have been taking place. The time lag of about two months between excavation and failure may have been associated with pore pressure equalization.

A Factor of Safety of 1.01 is obtained from a limit equilibrium analysis using strength parameters for the weathered mudstone and bentonitic clay found from laboratory tests. Pore water pressures observed in the field were used in the analysis. It should be noted that comprehensive case histories of first-time movements in fissured materials are rare (Skempton and Hutchinson, 1969) and it is of particular interest that the measured strengths, observed water pressures, and analysis give a consistent interpretation of the Devon Slide. The presence of both softened and unsoftened material in the back slope suggests that a decrease in shearing resistance due to weathering and an increase in water content is an important mechanism in the development of slides in this material.

The decrease of shearing resistance due to softening was considered in a similar manner for the Lesueur Landslide (Thomson, 1971) which failed in comparable material. The slope failure could be explained far more conclusively than in the previous investigation in terms of partly softened strength for the back slope materials

and residual strength along the bentonitic bedding plane. The configuration of the slide is shown in Fig. 4.11. The strength properties, as reported by Thomson (1971), as well as the modified values are represented in Table IV.4. The results of the stability analysis (Fig. 4.12.) suggest that the strength of the back slope materials at failure was governed by $\phi' = \phi'_{peak}$ and $c' \cong 340$ psf for the bentonitic clay, assuming that the residual angle of friction along the bedding plane was $\phi_r' = 10^o$. Because of the larger height of the slope the material appeared less softened and the strength less reduced than at Devon.

The distribution of both water pressures and strength parameters in the Devon Slide is extremely nonhomogeneous, but is not atypical of conditions in the Cretaceous bedrock of the Great Plains region of Western Canada and Northern United States. When such nonuniform conditions are encountered, detailed studies are needed to obtain knowledge of representative stratigraphy and the water pressure distribution. Unless continuous sampling is employed, critical bentonitic layers may be missed and unless the positions of piezometers are chosen with care, misleading information may be obtained. For example, if the water pressure distribution in the Devon Slide had been deduced from the level of water in the well close to the slide, a shear strength close to the peak value of the bentonitic clay would have been required to account for the instability. A piezometer near the slip surface was required in order

to determine the relevant water pressure. Furthermore, had borings not been undertaken outside the area of the slide, the sheared clay beyond the slide would not have been detected and the interpretation of the mechanisms causing the Devon Slide would have been altered radically.

2.2. Failure of Peace River Bridge at Taylor, B.C. (Hardy, 1966; Pennell, 1969)

In October 1957 the Peace River Highway Suspension Bridge near Taylor, B.C., failed 15 years after its completion, not because of structural defects, but due to geological conditions. The bridge failure was the subject of several investigations and has been described by Hardy (1966). Pennell (1969) quoted the slide as an example of progressive failure since the failure occurred within a little cemented shale at residual strength values. In the light of the results obtained from the present study it seemed very unlikely that progressive failure had caused the residual strength values. It rather appeared that the residual strength values had developed during previous old landslides which occurred when the Peace River was cutting its present channel. Hence a reinvestigation of the case was undertaken using the results of previous investigations, air photos, and local site inspections.

Geology

The Peace River district in the area is underlain by flat-

lying, nonmarine shales of the Fort St. John Formation of Upper Cretaceous age. They dip slightly southwards. The shales had been preloaded by several thousand feet of sediments and consequently unloaded due to erosion, which removed about 2000 feet of overburden. Surficial deposits, consisting of till, sands, gravels, and lacustrine silts and clays, cover most of the bedrock of the district (Jones, 1966).

The North side of the Peace River, on which the slide occurred, is characterized by a semi-circular high level gravel terrace about 200 feet above the level of the Peace River (Fig. 4.13.) with generally steep slopes towards the river. This terrace was formed subsequent to the last ice age. The gravel deposits, which are about 100 feet thick, overly the bedrock which is exposed along the North banks of the river and in some of the deeper ravines. The slopes of the Peace River Valley within the area are generally characterized by extensive slumping which was likely initiated when the river was eroding its present channel.

History and movements of slide

The Peace River Bridge was built in 1942 as part of the Alaska Highway. On the North side of the river the anchor block and the cable bent foundation were placed on bedrock, beneath the gravel layer.

In 1942 at the time when the bridge was built the main channel of the river was positioned along the South shore, but by

1948 it had shifted attacking the North bank over the length of the later slide (Hardy, 1966). Accompanying this shift in river channel scour developed below the North tower foundation requiring underpinning of the foundation in 1948. In 1952 the first evidence for horizontal movements of the North anchor block appeared. During 1956 and 1957 a natural gas scrubbing plant was built on the flat immediately North of the bridge. The water intake for the plant was built at the edge of the lower bench about 650 feet upstream from the bridge immediately to the left of the later scarp (See Fig. 4.14.). The pipe-lines from the intake to the plant were carried along the lower bench below the bridge North of the cable bent, and up the face of the bank about 200 feet East of the center line of the bridge. No excavation was made for the pipes along the bench. The gas scrubbing plant started operating in the summer of 1957.
In 1957 the total precipitation was about 80% above the average of the 35 years for which records are available; 70% of this occuring during the summer months (May to October) in 1957. Due to the surface run-off, which had been collected in the road ditches and discharged over the bank on either side of the anchor block, severe scour developed. In September 1957 there was visual evidence of movements. From 1952 to 1957 the top of the North cable bent had shifted southward by 1.08 feet, and the North anchor block had tilted with a horizontal movement of 1.6 feet.

"In October 15th by mid-afternoon a break in the lines of the water system of the gas plant was detected. At midnight it was clear that all lines were broken and the plant was shut down. For a period of several hours prior to the shut down the pumps were delivering 18 000 GPM water much of which was escaping into the bank and onto the lower bench only 200 feet East of the bridge. No marked deterioration in the bridge was detected until about 2.00 a.m. on October 16th. At this time a drop was discovered at the North end of the anchor block. This condition deteriorated rapidly and creep of the anchor block and cable bent foundation continued until the final collapse some eleven hours later." (Hardy, 1966)

The extent of the landslide is shown in Fig. 4.14., as reported by Moran, Proctor, Mueser, and Rutledge, 1958. The North cable bent pier had moved riverward by about 15 feet and in a westerly direction by about 4 feet. The North anchor block had moved riverward by about 10 feet.

Field investigations

Several series of borings had been made and test wells installed in the vicinity of the slide (See Figs. 4.14., 4.15., 4.16., 4.17.). These investigations included:

a) borings from which samples were obtained to determine the types and engineering properties of the subsoils;

b) exploratory borings to determine if zones of excessive pore water pressures existed;

c) test wells, some of which were pumped to decrease hydrostatic pressures;

d) slope indicator installations.

In 1967 block samples were collected of partially weathered shale to obtain residual strength parameters in terms of effective stresses and

pore pressures were measured (Pennell, 1969). In 1971 outcrops were studied by the author along the river bank as well as in a ravine of a small tributary.

The bedrock materials were described in detail by Moran, Proctor, Mueser, and Rutledge in 1958. An excerpt of their report is quoted in the following:

"General Characteristics

The shale below approximately Elev. 1270 to 1280, about 50 ft. to 80 ft. below the low terrace, is relatively unweathered. It is dark gray or black, often massive in appearance, horizontally bedded hard and has natural water contents of 3 to 9%; a landslide failure plane would not be expected to pass through such material. Shale closer to the ground surface exhibits weathering and is often a mixture of clay derived from the shale and of intact shale fragments. It may contain yellow or rust colored streaks, have water contents in excess of 10% and be much softer than the underlying shale. This weathered material is erratic in strength and in water content due to nonuniform weathering. The upper portion of this weathered zone, and some lower sections also, have changed into a clay soil which often is almost devoid of shale fragments and which gives little indication that it was derived from shale. The natural water contents are as high as 20 to 30%.

Significant Subsoil Characteristics

A zone of pulverized or fractured clayey shale at about Elev. 1290 was found in borings along the Alaska Highway Bridge." (See Figs. 4.14., 4.15., 4.16.) "The water contents in this zone are generally about 20% and are substantially higher than the water contents of the immediately overlying and underlying shale. This zone is described as pulverized clayey shale and as fracturey shale in the borings made for R. C. Thurber & Associates Ltd. ---

A zone of high water contents between about Elev. 1280 and 1290 also is found in several of the other borings as can be seen by reference to ---" (Figs. 4.15., 4.16., 4.17.) "Our examination of the sample sent to us from Boring 11AS at a depth of 52 ft., or approximately Elev. 1296, showed that it consists of clay and mixed up shale fragments having random orientations, ---. It appears to have been seriously disturbed and reworked by shearing probably due to a land-

slide. Another sample which exhibited serious disturbance apparently resulting from sliding was obtained at a depth of 65 ft., Elev. 1287, in Boring 5AS. Cur examination of the sample recovered at a depth of 80 ft., approximately Elev. 1278, from Boring 3AS showed that probably it has been disturbed by sliding. These borings are located within the slide area, which suggests that the disturbed samples and associated zone of higher water contents may represent the base of the landslide which occurred on October 15-16, 1957. However, they may also represent the sliding plane of a much older landslide which occurred when the Peace River was cutting down into its present channel. This possibility is based upon the zone of weathered shale found in Boring 6AS, outside the failure area, at about Elev. 1292. The shale at this depth was weathered into a clay having the relatively high water contents of 21 and 27%, ---." (See Fig. 4.15.)

"The above possibility is further indicated by the finding of gravel embedded in shale which is clearly foreign to shale. For example, in Boring 6AS a sample recovered from a depth of 65 ft., approximately Elev. 1293, contained quartzite gravel foreign to shale embedded in a soil which was clearly derived from shale. Except for the quartzite gravel, this sample appeared to be original material derived from shale through normal weathering. The remaining few shale fragments were easily broken by finger pressure. A penetrometer test on it indicated a shear strength of 0.7 ton/sq. ft. Boring 5WCT and several adjacent borings disclosed the presence of substantial amounts of gravel within strata of clay apparently derived from weathering of shale. This was also found in Boring 7RCT at the Alaska Highway Bridge. The gravel persumably was entrapped by old slides resulting from erosion at the toe of the north bank of the river. The age of the slides cannot be deduced other than that they are post-Pleistocene, as evidenced by the rounding of the gravel contained therein. The exposed shale bank immediately east of the slide shows unnatural mixed orientations of shale particles and foreign gravel particles embedded in the face. These suggest previous slides, but could conceivably represent local erosion and slope covering."

The investigation of the outcrops outside of the slide area showed further evidence of previous slides. Outcrops "A" were located in the ravine of a small tributary East of the gas scrubbing plant,

along a continuous break, which is visible on the gravel flat (See Fig. 4.13.) and is believed to indicate the boundary of a large old slide block: contorted gravel and silt beds could be observed just above a fault zone within the bedrock (See Fig. 4.18.). Further, bedrock could be found which was dipping North contrary to the general southerly dip of undisturbed strata.

Outcrop "B" was located in a small, rather shallow gully within the North bank of the river. A fault zone was indicated by a marker bed which was displaced by several feet. A number of springs was observed in the vicinity of the anchor block at the contact of the gravel and the shale (See Fig. 4.13).

Water levels measured during the summer 1967 indicated levels generally consistent with the bottom of the gravel layer. These levels were similar to those reported at the same time of the slide in 1957.

The samples from the test holes showed a range of natural water contents from 3% to 35%. Plastic Limits ranged from 16% to 25% and Liquid Limits from 25% to 48%. Intact shale showed in all cases natural water contents less than 9%. The undrained shear strength values c_u varied from 14 000 to 56 000 psf for samples at moisture contents below 9% to values of 500 psf for samples at natural water contents of 33%. In Fig. 4.19. the undrained shear strength values c_u are plotted against the respective natural water contents w_n. Triaxial compression tests indicated effective peak strength parameter

of $\phi'_p = 35^0$ to 37^0 and $c'_p \cong 10.4$ psi (Hardy, 1967). From the block samples the residual angle of friction was found to be $\phi'_r = 15^0$ to 16^0 (Pennell, 1969). The properties and parameters of the shale are summarized in Table IV.5.

Slope stability analysis

Pennell (1969) analysed the slide in terms of effective stresses, using Morgenstern and Price's slope stability program (Morgenstern and Price, 1965) on the basis of the following data:

1) The failure plane was located horizontally at elevation 1290 in the shale. Evidence for the failure plane was the zone of pulverized shale.
2) The piezometric head was located at the bottom of the gravel layer as measured in the test holes. The river level existed at elevation 1320.

When applying the residual parameters $c'_r = 0$ psf and $\phi'_r = 16^0$ Pennell found a Factor of Safety $F_s = 1.0$. For peak strength parameters the Factor of Safety was 3.3.

Concluding remarks

On the basis of the results from the stability analysis Pennell concluded:

"--- the fundamental internal cause of the slide was the decrease in strength of the clay shale from the peak to the residual."

The stability analysis shows that failure occurred at strength parameters which were close to residual within the shale.

It appears, however, that the slide was not caused by "the

decrease in strength of the clay shale from the peak to residual". The results of the investigation show quite unambiguously that the failure had taken place along an old slip plane. Evidence for previous sliding is given by

1) shear zones in boreholes outside of the slide area (Moran, Proctor, Mueser, and Rutledge, 1958),
2) faulted bedrock in outcrops below the gravel deposit,
3) air photos which suggest that the North abutment of the bridge is situated in an old slide zone. A break in the gravel surface possibly indicates the boundary of the old slide block. This assumption was supported by the observation of fault zones in bedrock outcrops right below this break.

Previous sliding probably occurred when the Peace River was cutting down into its present channel. It can be assumed that the present river banks are close to a metastable equilibrium. Relatively minor changes along the slope very likely reactivated movements. The bridge construction, the shift of the river channel, and finally the excessive precipitation were reasons enough to cause a new failure. Further softening of the back slope might have happened resulting in a deterioration of cohesion.

The delay of 15 years can be explained by the time dependent accumulation of the above factors, but could also be due to the slow equalization of pore pressures. Unloading of the slope during

construction of the approach road to the anchor block might be reflected in the effective stress conditions only at a later date.

2.3. Bjerrum's Cases

Bjerrum tried to prove his theory of progressive failure by a number of case histories, in which failures were described at strength values near residual. These cases will be discussed in the light of additional information which could be obtained more recently. It will be shown that none of the quoted case histories constitutes a conclusive example of his mechanism affecting c' and ø'.

Slide in Sandnes, Norway, (1963)

The landslide in Sandnes occurred in 1963 within an area which formerly was a brick clay-pit that had been abandoned in 1952. The original geological conditions before the clay-pit was excavated are shown in Fig. 4.20. Below a blanket of sandy till were found in succession three horizontal layers of interglacial overconsolidated clays of high strength. The top layer was a sandy low plasticity clay, overlying a 30 to 45 feet thick deposit of stiff fissured plastic clay, with a water content just above the Plastic Limit. The lower clay was characterized by a high water content, which was approximately equal to the Liquid Limit. Embedded in the lower silty clay was a continuous sand layer with a thickness of about 2 feet. The sand layer was assumed to communicate with the groundwater in the backland and hence caused artesian conditions near the toe of the slope.

The slope was reported intact in the middle of the last cen-

tury with no signs of slides or movements; the average inclination of the slope was then 5 horizontal on 1 vertical.

In 1878 a brick works was built at the toe of the slope and production was started using the sandy clay excavated from the slope. Around 1900 when the sandy clay had been removed and the plastic clay became exposed, a deep seated failure was initiated, which started with a bottom heave. As excavation of the clay at the toe of the slope continued, successively greater and greater parts of the slope became involved in the slide (See Fig. 4.21.). Over the years, a procedure of excavation was adopted which made full use of the deep seated movements. Over the summer period clay was excavated in a pit at the toe of the slope. During winter excavation was stopped and in spring when the work was resumed the pit had filled itself with failed mass. When brick production was abandoned in 1952, the slope had been disturbed by a succession of slides (See Fig. 4.21.).

In 1952 the area was bought by the city of Sandnes for an expansion of the residential area, after the possibility of deep seated slides in the clay had been excluded. In 1955 the grading of the area was started. In the fall of 1956 after a period of heavy precipitation a crack was observed which approximately followed the scar of the later failure.

In August 1963 a relatively shallow excavation at the toe of the slope, which exposed the plastic clay, triggered a large slide. A stability analysis showed that the strength along the failure zone

within the plastic clay was close to residual. The clay had already been strained so far beyond failure by the previous landslides during the clay mining operation that no further reduction of strength would be expected. Consequently the movements could be stopped by relatively modest means.

Bjerrum's explanation of the slide from August 1963 was thus that

> "by progressive failures a continuous sliding surface had developed along which the shear strength was reduced to its residual value."

The author believes that the continuous sliding surface had already developed before, as a result of the many slope failures which had happened during the clay mining operation. These slides again had been caused by the unloading of the toe of the slope during excavation. The time lag between excavation in the fall and failure during the winter had very likely been caused by groundwater changes or softening.

Excavation for Freeway in Seattle, USA, 1962-1963

(Wilson, 1964; Palladino, 1971)

During the construction of the Seattle Freeway cuts were excavated which caused serious stability problems. In most cases a dense glacial till was overlying a deep formation of lacustrine, overconsolidated medium plastic clay of high strength. The clay is partly varved and exhibits a system of fissures and joints. Particularly, cuts excavated through the glacial deposit into the plastic clay caused serious

stability problems. Many of the slides could be explained only in terms of residual strength along the horizontal parts of the slip plane.

Bjerrum suggested that these low strength values were

> "excellent examples of the development of a slip plane by progressive failure resulting from the removal of the lateral support on an unweathered overconsolidated plastic clay."

More recent investigations (Palladino, 1971; Hendron, 1971), however, revealed the presence of pre-existing sliding surfaces in the clay, along which residual strength values were obtained. The failure surfaces had been referred to the almost complete reduction in the lateral support of the valley slopes upon deglaciation. Slip planes also could have developed during glacial ice movements.

The Culebra Slides, Panama, (USCE, 1970)

Until recently the large failures along the Panama Canal could only be explained in terms of residual strength along the horizontal zones of the slip plane. These strength characteristics, as well as the sometimes substantial time lag between excavation and failure, made Bjerrum believe that these slides could have been caused by a mechanism of progressive failure. In recent more detailed investigations the shear strength was measured (Banks, 1972) and pore pressure readings had been obtained, which showed that in many places the pore water pressures were below the canal water level (USCE, 1970). These observations indicated completely different conditions than previously assumed. The strength criteria suggested in 1947

(MacDonald, 1947), e.g. $\phi = 15^{o}$ and $c = 1.15$ tsf, were found to be very conservative for slopes in the order of 600 feet but not for slopes about 300 feet and lower. From large shear tests in the field as well as from direct shear tests on slaked Culebra Shale zero cohesion and peak angle of friction ($\phi' = 19^{o}$) were found to be the governing strength parameters (Banks, 1972). Stability analysis for first-time slope failures at the East and West Culebra Slides indicated field shear strength of $\phi' = \phi'_{peak}$ and $c' = 0$. The time lag between excavation and failure possibly can be attributed to the slow dissipation of the negative pore pressures in the slopes, which had been caused by the unloading process during excavation. Also strength loss during softening can add to the delay of failure.

Slides in the banks of the river Volga, USSR (Popov, 1951; Zaruba and Mencl, 1961)

The geology of the wide plains between Gorki and Stalingrad in the USSR is characterized by sandy Tertiary deposits which are underlain by Jurassic shales and covered by glacial and alluvial sandy materials. The river Volga has cut itself through the upper cover of more or less sandy rocks and soils, forming steep banks along both sides of the river. Where the river has cut into the underlying Jurassic shales, huge slides occurred and are still occurring. The slides were reported to happen mostly along a horizontal plane. Bjerrum explained these slides in terms of progressive failure caused by the horizontal expansion of the shale (See Fig. 4.22.).

The slides along the banks of the river Volga appear very similar to the slides which are characteristic of the river valleys in the Great Plains of North America, and they probably can be explained in a similar manner. Pre-existing sliding planes along bedding planes, particularly along the interface between softer and harder materials are common features in the argillaceous sediments of the plains (Matheson, 1972). Daylighting of these zones of weakness by the erosion of the river very likely caused failure of the slopes along these zones. A study of the landslide distribution along rivers in Alberta (1972) also indicated a strong correlation to the position of the groundwater table.

Slope failures at South Saskatchewan Dam (Ringheim, 1964)

In connection with the excavation for the South Saskatchewan Dam project in Canada, various slope failures occurred in the banks of the South Saskatchewan River within weathered Bearpaw Shale. It was reported that most of these failures had been the direct result of stress changes caused by the excavation. The planes of movement were always near the bottom of the weathered zone and in almost all cases nearly horizontal. Residual strength was the governing strength along the horizontal slip planes. Bjerrum explained

> "that the movements were a result of an energetic tendency of the upper weathered clay to expand laterally, proving that the clay must have possessed a large amount of recoverable strain energy. --- the excavation gave the clay an opportunity to expand laterally, leading to the development of a continuous sliding surface by progressive failure."

It appears clear, however, that these failures are not examples for progressive failure since they were clearly reactivated slides. The banks of the South Saskatchewan River are characterized by extensive old landslides which had occurred along horizontally bedded strata and had reduced the strength to residual along these planes. Since these slopes are metastable, relatively small changes within these zones can cause further movements.

The point that all failures had occurred only within the weathered zone, can be explained by the observation that the regions of old landslides are commonly strongly weathered as a result of the disturbances which had occurred during sliding (Esu, 1967; Nakano, 1967). As was shown previously weathering decreases the strength of a soil, which also might have augmented a tendency for instability.

Slope failures at Balgheim, Germany, 1957 (Einsele, 1961)

Shortly after the excavation of a trench for a water line two slides occurred in weathered colluvial material which was overlying a heavily overconsolidated shale of Jurassic age ("Braunjura γ"). The slides were about 250 metres apart, situated at the same level of a rather gentle mountain slope. The one slide was found to have failed along a pre-existing shear plane which was located right on the interface between weathered colluvium and unweathered bedrock. The second slide (See Fig. 4.23.) also failed along the same boundary of materials; however, no slip plane could be detected outside the failure zone. It was concluded that the plane of failure was not a

pre-existing slip surface. Slickensides which could be observed within the slope material showed a very different inclination from that of the failure plane and thus could not be correlated with the failure. The mobilized shear strength along the failure surface was found to be lower than predicted from laboratory tests for both cases. Below the location of the second failure three terrace-like features had been reported. Because of their regularity the possibility of lobes resulting from earthflows was excluded and they were interpreted as man-made, old agricultural terraces. However, there was clear evidence from the mineralogy of the weathered colluvial material that it had been carried down the mountain slope during the Pleistocene by periglacial earthmovements and mudflows. Underneath solifluction sheets, continuous slip planes should be expected (Skempton and Petley, 1967; Weeks, 1969).

The pre-existing slip plane which had been observed for the first failure could be explained by the periglacial movements. Since the colluvial material basically was the same at both locations it can be argued that the second slide, being only 250 metres away, also occurred along an old slip plane.

Slide at Jackfield, England, 1952 (Skempton and Henkel, 1954; Skempton, 1964)

The landslide at Jackfield involved the weathered zones of the valley slopes of the river Severn, England. It had been investigated and described by Skempton and Henkel (1954) and was referred to

again in Skempton's Rankine Lecture (1964). The slopes in which the slide occurred were reported as being covered by a mosaic of landslides of varying ages. Therefore it is clear that the present failure occurred along the slip plane created by the previous landslides. Hence this case history is not useful in studying quantitatively mechanisms of progressive failure.

2.4. Other Case Histories

Far more studies of landslides exist which have been reported as failures at strength values close to residual and consequently have been explained by some mechanism of progressive failure. Some of the more detailed reports will be discussed in the following.

Gradot Ridge landslide, Macedonia, 1956

(Šuklje and Vidmar, 1961; Skempton and Hutchinson, 1969)

A very large catastrophic landslide occurred in September 1956 in the West slope of the Gradot Ridge, Macedonia, filling the valley of the Vatasha River up to a depth of 70 metres (See Fig. 4.24). The slope had failed along the top of a stiff lacustrine clay of Pliocene age, which was overconsolidated by the erosion of the valley. The clay is overlain by a thick bed of well compacted silt, followed by weakly cemented sandy gravels and capped by an andesite tuff. The strata are dipping at 4^0 towards the valley. The soil properties are summarized in Table IV.6. Along the upper boundary of the slide a discontinuous series of deep cracks had been observed for at least 50 years. The largest crack was about 20 metres long, 1.30 metres

wide and more than 30 metres deep. A year before the failure the top of the future slide subsisted by 0.2 metres. Further evidence of movements was reported some weeks before the final collapse.

The slide area is situated in a zone of severe earthquakes. A stability analysis of the slide, assuming a continuous crack through the tuff cap and zero cohesion and peak friction angles along the entire slip plane, resulted in an average Factor of Safety F_s = 1.14. This suggested that the strength parameters mobilized during failure along the clay zone were c' = 0 and $\phi' < \phi'_{peak}$. Skempton (1969) thus proposed that some progressive failure had reduced the angle of friction to a value less than peak.

The cracks observed within the slope long before the slope failure indicate that the slope might have been always close to a metastable equilibrium. Therefore it can be assumed that the strength along the clay-silt interface was always close to the strength value which had been activated during failure. Reduced strength along bedding planes, especially along the interface of softer and harder layers have been commonly observed (Schultze, 1956; Dolezalek and Duero, 1962; Esu and Calabresi, 1969; Conlon and Tanner, 1971). Zero cohesion, possibly even a reduced friction angle, could be the governing strength parameters. The frequent earthquakes might have further reduced the strength within the slope, not only along the clay-silt interface, but also within the back slope materials by overstraining. In addition softening processes might have reduced the strength of the back slope

starting along the observed cracks. A further reason for the decreasing stability of the slope could have been the progressive erosion of the Vatasha River. It was reported that the movments started at the northern part of the slope, where the river had already exposed the clay layer along which the slip occurred.

The slide can be explained as a failure along a zone of weakness triggered by the summation of the above factors. Considering the crudeness of the investigation the Factor of Safety of 1.14 is already indicative for instability, and cannot be conclusively used as an argument for progressive failure. Particularly, the water pressures, which were assumed according to wells in the slope, are very likely only rough approximations of the true conditions.

Failure in an open pit mine, Germany, 1966

(Breth and Wanoschek, 1970; Wanoschek and Schmidt, 1970)

In November 1966 the slopes of a waste tip failed within an open pit mine which had been abandoned some 50 years ago. Since its deposition no movements had been observed and it had been considered stable until six days before the failure, when the first cracks were observed. No changes of the fill had been reported. However, the water table of a pond situated at the foot of the slope had risen constantly by 2 metres within the three years previous to the slide (See Fig. 4.25.).

The fill of sandy, gravelly waste materials had been placed on top of a thin layer of coal, which was overlying a stiff medium

plastic clay. The clay layer was dipping at approximately 2 degrees towards the excavation into which the slope had slid. The soil properties are summarized in Table IV.7. From the test results and the slide behavior it had been assumed that the slip occurred along the coal clay interface within the clay. The slide stopped after it had moved about 20 metres.

Six piezometers were installed after the slope failure and the pore water pressures have been measured over a period of one year. The measured water levels are indicated in Fig. 4.25. The strength parameters of the fill were found to be $\phi' = 35^{o}$ and $c' = 0$ and for the coal $\phi' = 31^{o}$ and $c' = 1\ t/m^2$. The clay was tested in ordinary shear tests and in the "Vienna Routine Shear Test" (Borowicka, 1963). A surprisingly low angle of friction at peak of 16^{o} was reported. The residual angle of friction was after very large strain $\phi'_{residual} = 6^{o}$. Also this value appears very low, when compared with the Plasticity Index obtained for the clay (Bjerrum, 1967). In the consolidated undrained triaxial tests with pore pressure measurements very little reduction of strength after peak could be observed even at large strains.

The cohesion had been assumed zero. They attributed this reduction to the unloading during the mining operation. The influence of the variation of the groundwater table on the slope stability has been investigated. In Fig. 4.26. the slope is shown, and for several groundwater tables, the corresponding critical failure surfaces and the corresponding average angle of friction at failure along the clay inter-

face are indicated. It can be noted that after the rise of the water table in the pond a markedly higher strength along the clay-coal interface was needed for stability than at the previous lower water table. For groundwater condition "D" failure would have occurred at $\phi'_{mobilized} \cong \phi'_{peak} = 17^0$, while for groundwater condition "B" failure would have occured at $\phi'_{mobilized} \leq 10^0$ which is already close to the residual angle of friction. Taking in account the change in geometry the failed slope was found to be stable at an effective angle of friction along the coal-clay interface of $\phi' = 7^0$ to 10^0.

From the low angle of friction after failure as compared with the peak value the authors of this study concluded that a mechanism of progressive failure had caused the slope to fail. It was believed that the increase of the water table had caused the shear strength along the coal-clay interface to pass the peak at one point and that consequently failure began spreading progressively from this point. However, the very flat-topped stress-strain curves of the clay which had been obtained from the triaxial tests, indicate that progressive failure would be very unlikely in this kind of clay. Extremely large strains are needed to cause a marked reduction in strength. The low angles of friction which had been computed for the slope after failure could develop only following very large strains, which in fact had taken place during the slope failure. Movements along the base of more than 20 metres were reported.

The clay showed the strength characteristica of a normally

consolidated or remoulded material probably due to the presence of joints, fissures and slickensides. Discontinuities within the clay have not been reported in this study. However, slickensides have been widely described as a characteristic feature of underclays, particularly for the underclays of the North West German brown coal regions (Dolezalek and Duero, 1962; Schultz, 1958). Some authors (Schultze, 1956) could even measure effective angles of friction along the interface between coal and clay which were substantially below the peak value of the clay. Instability could also be explained by the rise in water table. For a groundwater table as assumed by situation "D" (See Fig. 4.26.) the slope was found instable for an angle of friction close to peak. The time gap of several months between increase of water level and failure can be accounted for by delayed pore pressure equalization.

Failure in a slope of overconsolidated plastic clay, Germany, 1961 - 1969 (Wanoschek and Schmidt, 1970)

In 1961 a slope started to slide which in 1955 had been steepened by a fill on top and a cut on the toe (See Fig. 4.27.). The inclination of the original natural slope was around 8 degrees. The slope material was a stiff fissured Tertiary clay of high plasticity. The soil properties, varying with depth, are summarized in Table IV.8.

The clay within the failure zone was found markedly softer than the clay from below. From triaxial tests the angle of friction

was found to be $\phi' = 24.5^o$ for the materials from below the failure zone and $\phi' = 9.5^o$ for the materials from the sliding zone.

Because of the low angle of friction in the failure zone and the time lag between excavation and failure the slide was described as progressive failure. It appears, however, that the slide was simply caused by the loading of the top and the unloading of the toe of the slope. The delayed failure can be referred to the slow dissipation of the excess pore pressures. The failures were generally very shallow and were restricted to the weathered zone. The flatness of the natural slope as well as its hummocky surface suggest that the slope had been affected by old shallow landsliding or periglacial solifluction as had been observed for London Clay (Weeks, 1969; Skempton and Petley, 1967). These periglacial movements would explain the low values of internal friction which had been measured for the clay from the slide zone.

3. Slope Failures in Materials with Unstable Stress Strain Curves, where the Average Shear Strength Mobilized along the Failure Surface was at Peak

Slope failures where an average shear strength along the failure surface close to peak was mobilized in spite of unstable stress strain properties of the failed materials indicate insubstantial influence of the effect of nonhomogeneous stress distribution along a potential slip plane on the stability of slopes (Turnbull

and Hvorslev, 1967). Nonhomogeneous stress distribution can be expected to occur in any slope within overconsolidated materials. Thus, if a nonhomogeneous stress distribution does in fact cause locally reduced strength values, there should be no slope failure where the mobilized average shear stress along the failure surface is equal or close to peak strength.

There are a number of well documented cases for which failures at peak strength values had been observed, such as the slides in Lodalen (Sevaldson, 1956), Selset (Skempton and Brown, 1961; Skempton, 1964), along the Kimola Canal (Kankare, 1968), Selnes (Kenney, 1968), and in the Welland Test Pit (Kwan, 1971). These examples, however, cannot be considered as conclusive arguments against nonhomogeneous strength mobilization, because in all these cases the materials had non-brittle and almost flat-topped stress strain curves. Here substantial strains result in small strength reduction. Nevertheless, a rather substantial drop of strength from peak to residual can be expected, after extremely large shear strains, probably in the range of 20 to 30 percent. The flat-topped stress-strain curves also inhibit softening.

It is of interest to indicate how flat-topped the stress strain curves have to be so that strength reductions during pre-failure movements are unlikely to occur. In order to describe the shape of the stress strain curve quantitatively it is convenient to introduce an index which compares the work done during stable yield-

ing and unstable yielding (See Fig. 4.28.). The Work Softening Index describing the flatness of the stress strain or load deformation curve is defined as:

$$I_W = \frac{A_2}{A_1}$$

in which A_1 = work done during stable yielding, as described by the area between the points A-B-C-E

A_2 = difference of work done during stable and unstable yielding, as described by the area between the points B-C-D.

This new index appears preferable to the Brittleness Index I_B and the Rupture Index I_p as defined by Bishop (1967). The Brittleness Index compares peak strength with residual strength values. The above materials, however, indicate substantial differences between peak and residual strengths, and therefore cannot be differentiated from materials with distinct peaks on the basis of the Brittleness Index. The Rupture Index like the Work Softening Index describes the stress-strain curves in terms of strain energy, but considers only the post peak behavior of a soil and hence ignores the ductility. The Work Softening Index is concerned with the complete stress strain behavior.

For most materials with very flat-topped stress strain curves data is incomplete and the shape of the curves have been estimated as shown in Fig. 4.29. The residual frictional angle could be obtained from the Plasticity Index of the material as suggested by Bjerrum (1967). The Work Softening Index was computed for stress strain

curves at stress levels representative for the landslide.

In Fig. 4.30. the Work Softening Index is plotted against the Factor of Safety for actual failure computed using peak strength parameters. It may be noted that for materials with $I_W < 0.2$ the Factor of Safety in terms of peak strength was around 1. Beyond the value $I = 0.2$ the Factor of Safety in terms of peak strength increased rapidly with increasing I_W - values. This illustrates the magnitude of strength reduction that might occur before failure in soils with unstable stress strain curves. It can be concluded that in slightly overconsolidated non-fissured clays with flat-topped stress strain curves, as described by $I_W < 0.2$, the likelihood of progressive failure in practice is small. Therefore slopes in these materials might be designed in terms of peak strength even for long-term conditions. The presence of open fissures, however, could lead to Terzaghi-softening and resultant strength reduction, even in this kind of materials.

For the material of the slopes of the upper Kimola Canal at Section 43+00 (Kankare, 1968) the Work Softening Index was $I_W = 0.35$, which is indicative of the possibility of some form of progressive failure. However, the slopes have not failed so far, although several smaller slips occurred in the vicinity. By using Bishop's simplified method for a circular arc and applying the most critical pore pressures which had been measured a Factor of Safety was found to be 1.17 in terms of peak strength. In terms of fully softened strength parameters ($c' = 0$ and $\phi' = \phi'_{peak}$) the Factor of Safety was 0.76. No progressive

failure due to nonhomogeneous stress distribution has occurred so far, again indicating the limitations of this mechanism. However, if joints have opened during the deformations which might have occurred, Terzaghi-softening can cause strength reductions and possibly failure of the slope some time in the future.

TABLE IV.1

DEVON SLIDE: RESULTS OF TRIAXIAL TESTS ON SLIP SURFACE MATERIAL

Pit No.	Sample No.	Water Content Before Test	Water Content After Test	σ_3'	$(\sigma_1 - \sigma_3)$	Angle of Failure Plane to Horizontal	ϕ' residual
		(%)	(%)	psi	psi	degrees	degrees
3	2	35.3	45.53	30	60.0	8.0	5
3	3	38.3	48.78	30	40.3	20.5	10
3	4	32.3	50.90	30	28.7	20.0	9
Average							8

TABLE IV.2.

DEVON SLIDE: RESULTS OF TRIAXIAL TESTS ON BACK SLOPE MATERIALS

Test No.	BH	Level A.S.L.	W.C. Before Test	W.C. After Test	Effective Consolidation Pressure	σ_3' Failure	$(\sigma_1 - \sigma_3)$	Angle of Failure Plane to Horizontal	Strain to Failure
		(feet)	(%)	(%)	psi	psi	psi	degrees	(%)
1	5	2212	35.1	28.0	10	10	30.0	45	6.5
2	1	2221	27.2	21.8	50	50	55.0	45	2.3
3	5	2211	28.4	22.5	5	3	12.0	50	6.1
5*	1	2209			30	17	49.0	30	6.0
6	5	2209	25.9	23.5	30	23	55.0	57	4.1
4	1	2217	21.5	18.5	15	19	90.8	57	5.0
7	1	2200	20.3	17.0	10	- 9	63.0	50	4.9

* contains Bentonite	50.0	41.8
Brown Shale	31.8	26.0
Sandstone	19.6	17.8

[illegible]

[illegible]

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

TABLE IV.3.

DEVON SLIDE: SUMMARY OF INDEX AND STRENGTH PROPERTIES

Material	w_N	w_p	w_L	I_p	I_L	%<2μ	c'	ϕ'	c' residual	ϕ' residual
	(%)	(%)	(%)	(%)		(%)	psf	degrees	psf	degrees
Bentonitic Clay	48	39.8	100	60.2	.136	47			0	8
Weathered Back Slope Clayey	31.8	20.1	84.8	64.7	.181	52	190	33		
Weathered Back Slope Sandy Silty	26.6	20.6	44.9	24.3	.247	32				
Unweathered Back Slope Sandy Siltstone	20.9	20.5	43.0	22.7	0	22	3720	19		

TABLE IV.4.

LESUEUR SLIDE, CANADA: SUMMARY OF STRENGTH PROPERTIES

	Strength Parameters after Thomson, 1971				Strength Parameters used in Analysis		
	c'_{peak} psf	ϕ'_{peak} degrees	$\phi'_{residual}$ degrees	γ_{total} psf	c' psf	ϕ' degrees	$\phi'_{residual}$ degrees
Silt	0	41	38	112	0	41	--
Till	600	24	23	132	400	24	--
Sand and Gravel	0	38	35	120	0	38	--
Hard Bentonitic Clay	865 to 1300	14 to 24	10	113	varied	33*	varied

* as measured in Devon Slide

TABLE IV.5.

PEACE RIVER BRIDGE SLIDE: SUMMARY OF SOIL PROPERTIES (AFTER PENNELL, 1969)

Index Properties

w_L	w_p	I_p	A	w_N	e_N	I_L	γ_t	%sand	% silt	%clay	G_s
%	%	%		%			pcf	%	%	%	
37	20	17	.5	14	.59	-.35	128	10	56	34	2.67

Mineralogical Composition of Clay Fraction

Montmorillonite %	Illite %	Kaolinite and Chlorite %
2	73	25

Shear Strength Results

ϕ'_{peak} degrees	c'_{peak} psi	$\phi'_{residual}$ degrees	$c'_{residual}$ psi
30 to 37	0 to 10.4	15 to 16	0 to 1.6

TABLE IV.6.

GRADOT RIDGE LANDSLIDE, YUGOSLAVIA: SUMMARY OF SOIL PROPERTIES

Soil Properties after Suklje and Vidmar (1961)						
	$\% < 2\mu$	w_l (%)	I_p (%)	ϕ' peak degrees	ϕ' residual degrees	ϕ' *) residual degrees
Clay near slip plane	20	62	35	24.5	15.5	12
Clay above slip plane	10	50	28	21.0	19.0	16
Silt	5	35	10	28	18.0	26
Sandy gravel	--	--	--	38	--	--
Tuff	--	--	--	--	--	--

*) Estimated values from ϕ'_r - I_p relationship (Bjerrum, 1968)

TABLE IV.7.

SLIDE IN OPEN PIT MINE, GERMANY: SOIL DATA

(AFTER BRETH AND WANOSCHEK, 1970; WANOSCHEK AND SCHMIDT, 1970)

	w_N	w_L	w_p	I_p	$\% > 2\mu$	Mineralogy Quartz	Kaolinite	Illite	Montmorillite	ϕ' peak	c' peak	ϕ' residual
	%	%	%	%	%	%	%	%	%	degrees	Mp/m^2	degrees
Fill	--	--	--	--	--	--	--	--	--	35.0	0	--
Coal	--	--	--	--	--	--	--	--	--	31.0	1.0	--
Clay	16.4-18.7	48.4-62.1	18.4-24.6	28.4-37.5	54.0-64.0	50	25	25	0	16.0	2.0	6.0

TABLE IV.8.

SLIDE IN OVERCONSOLIDATED PLASTIC CLAY

(AFTER WANOSCHEK AND SCHMIDT, 1970)

Depth Metres	w_N %	w_L %	w_p %	I_p %	A	ϕ' peak degrees	ϕ' residual degrees
1.9	33	77	24	53	1.16		
2.2	24	70	23	47	1.21		
3.7	21	69	23	46	1.10	18 to 14	5 to 7.5
6.0		87	26	61	1.10		

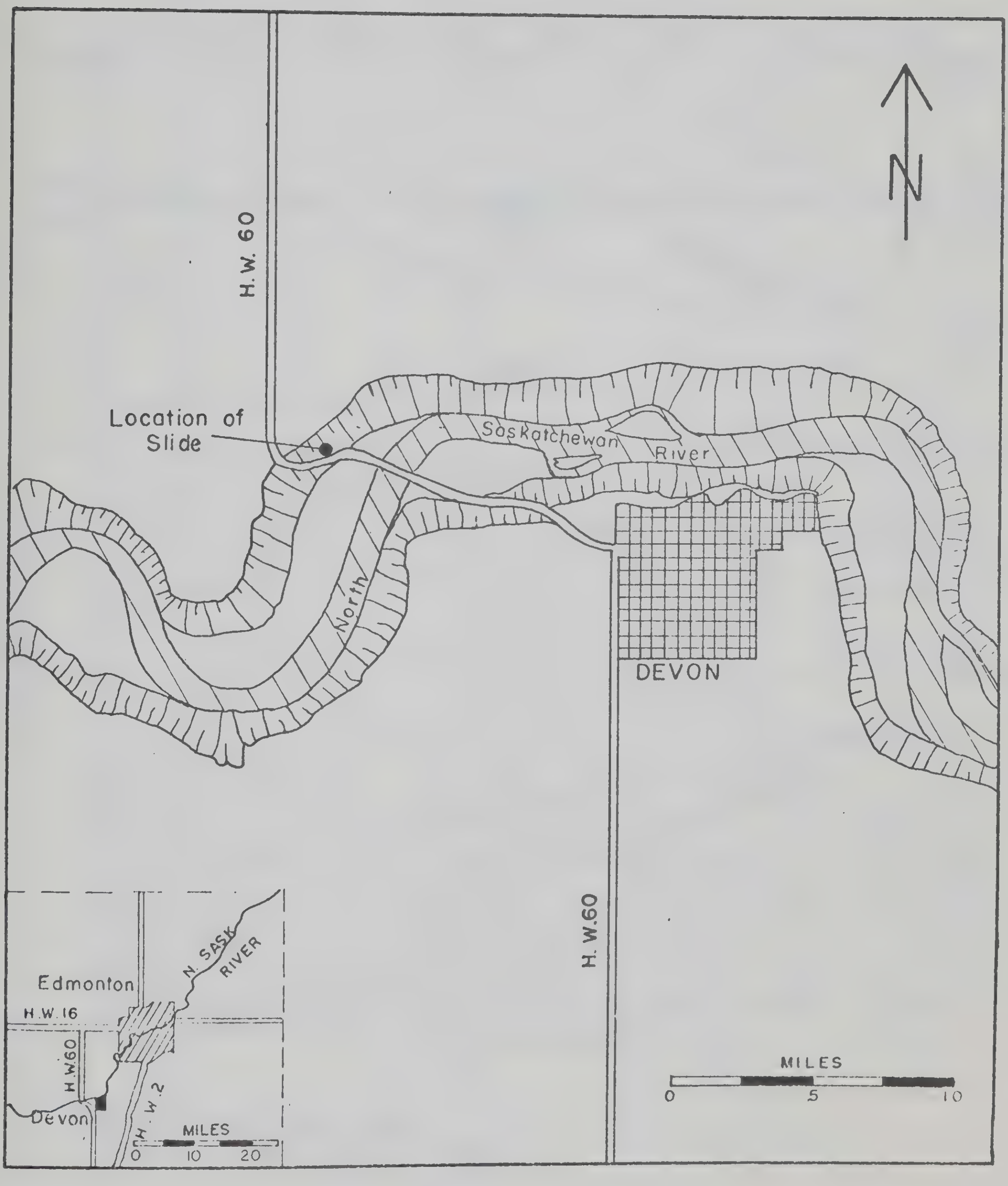


FIG. 4.1. DEVON SLIDE:
MAP SHOWING LOCATION OF LANDSLIDE

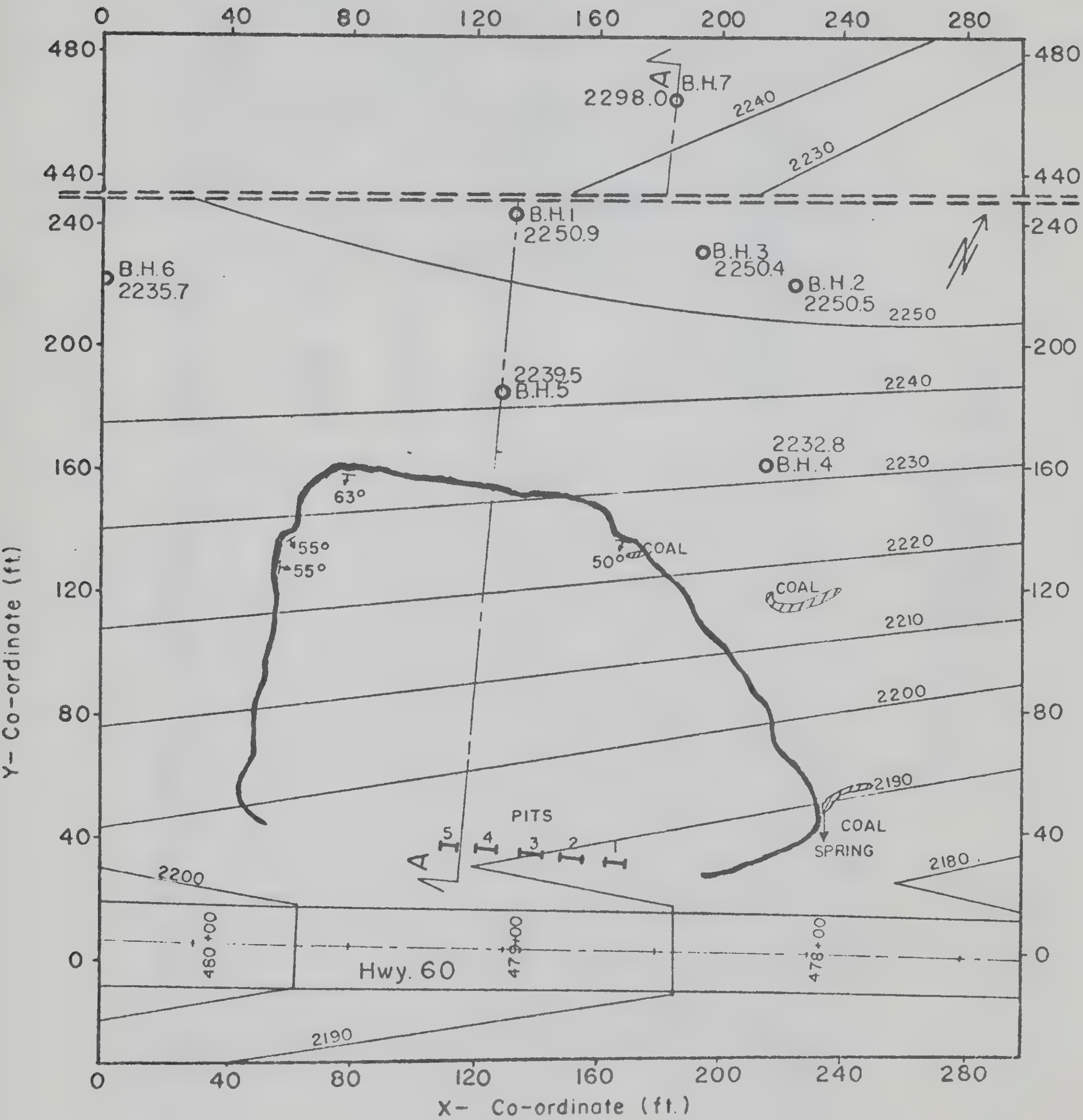


FIG. 4.2. DEVON SLIDE:
MAP OF SLIDE AREA

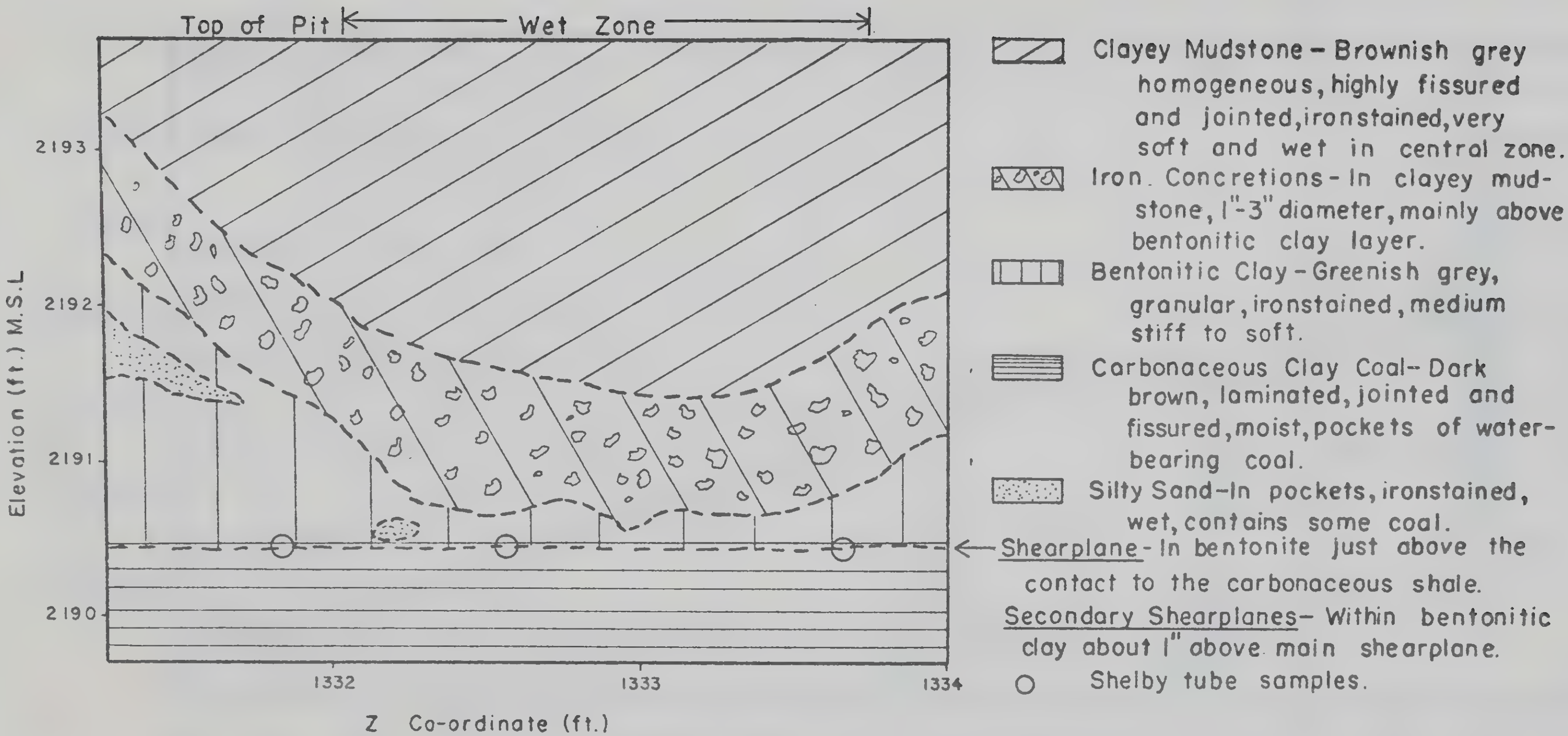


FIG. 4.3. DEVON SLIDE:
SKETCH OF PIT NO. 3

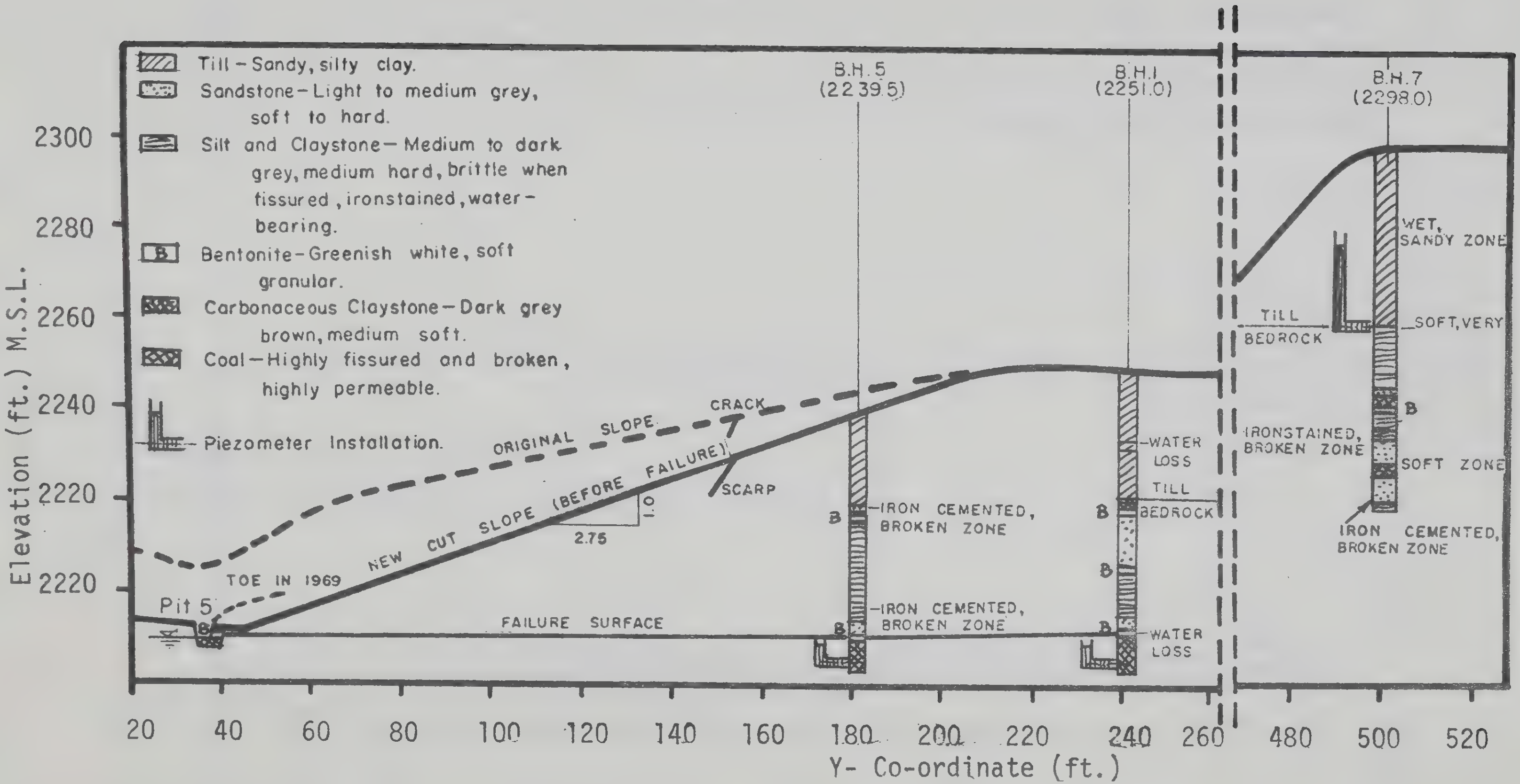


FIG. 4.4. DEVON SLIDE:
CROSS SECTION A - A

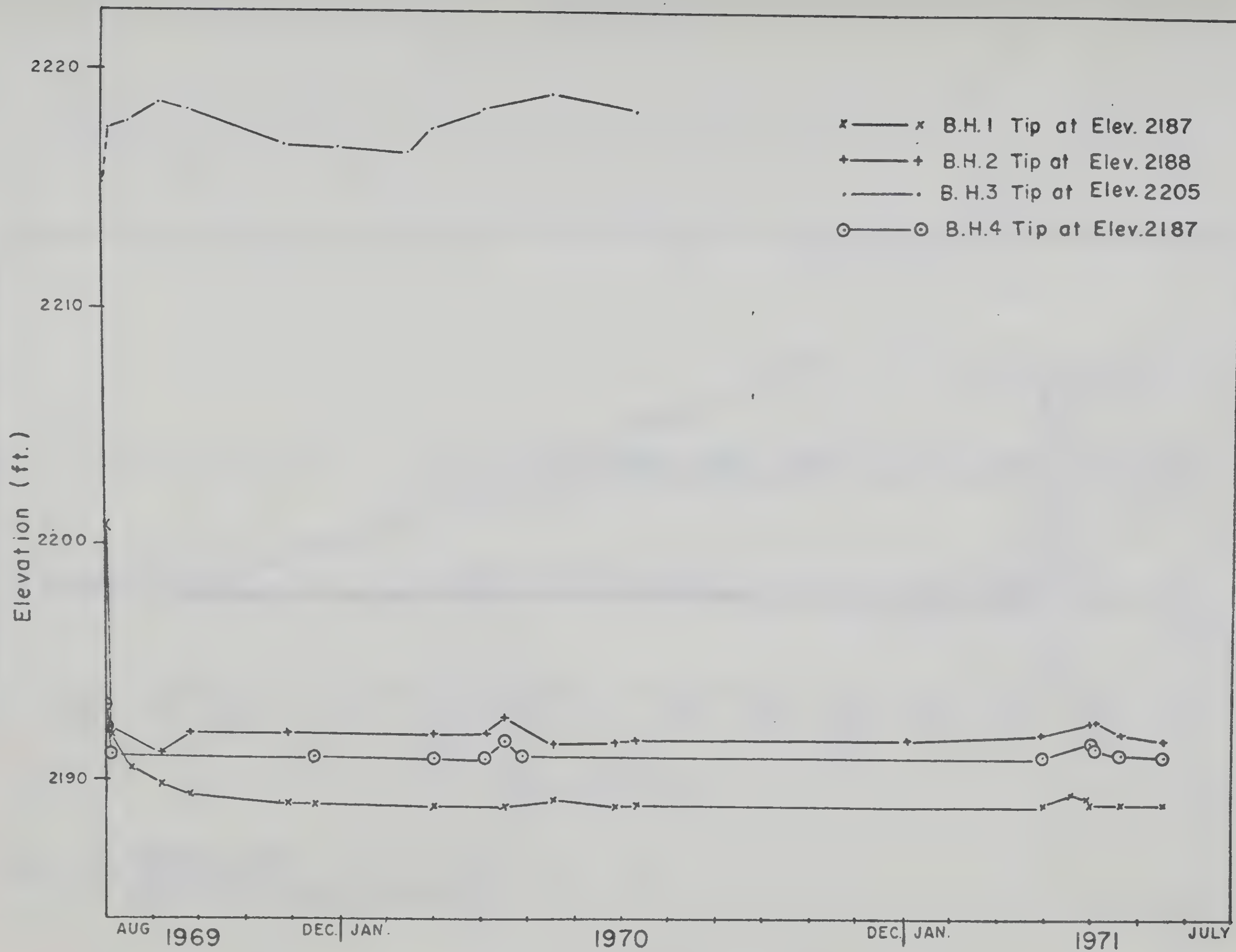


FIG. 4.5. DEVON SLIDE: PIEZOMETER READINGS IN BOREHOLES

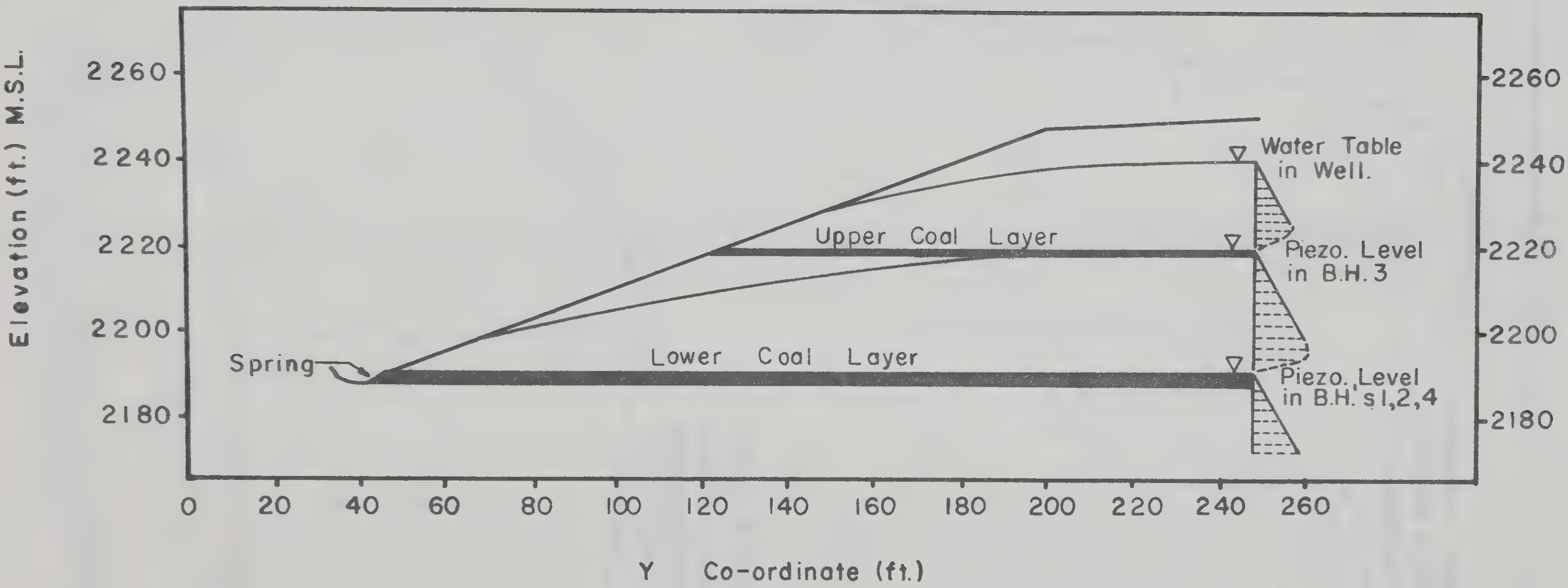


FIG. 4.6. DEVON SLIDE:
WATER PRESSURE DISTRIBUTION

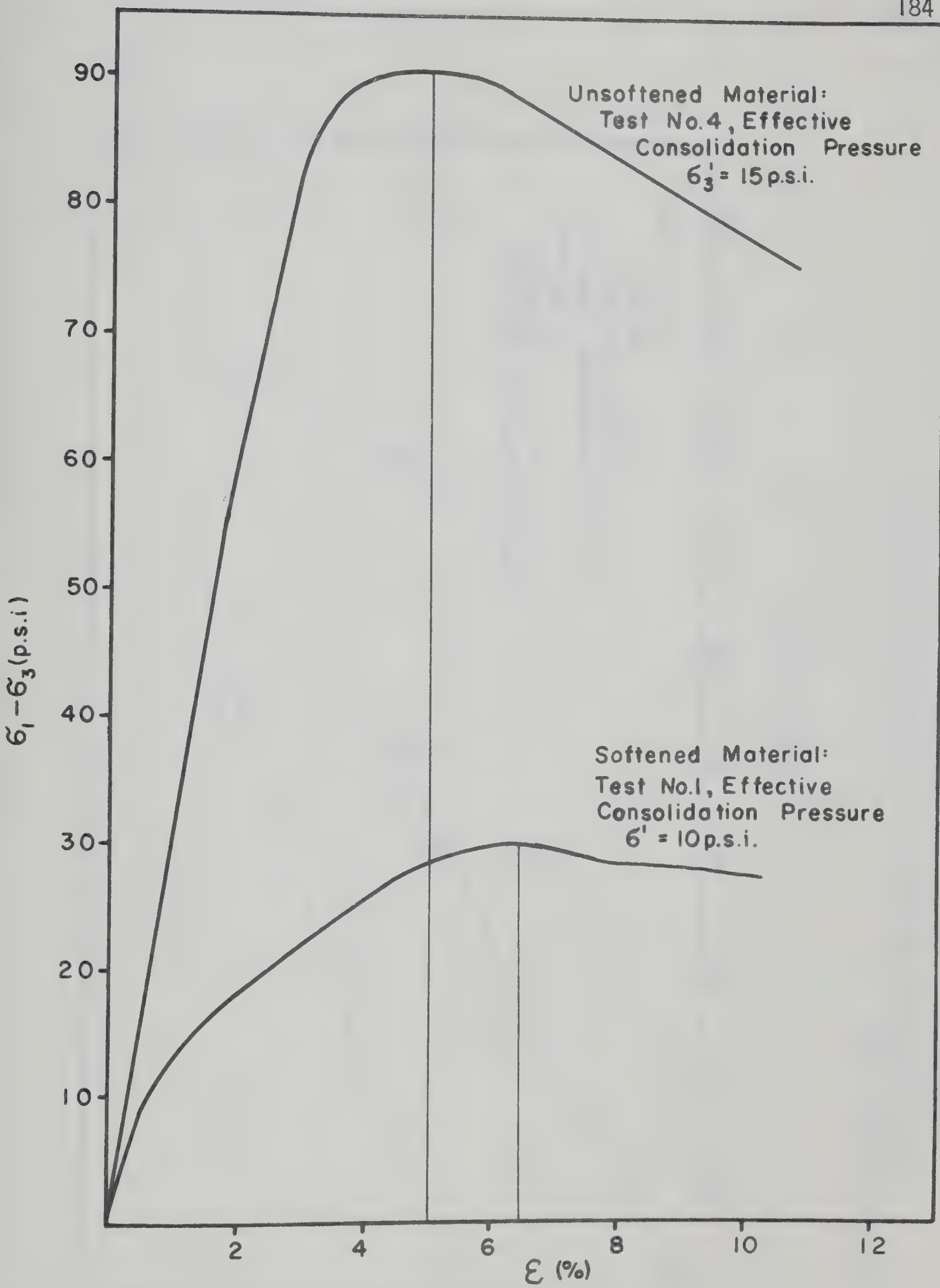
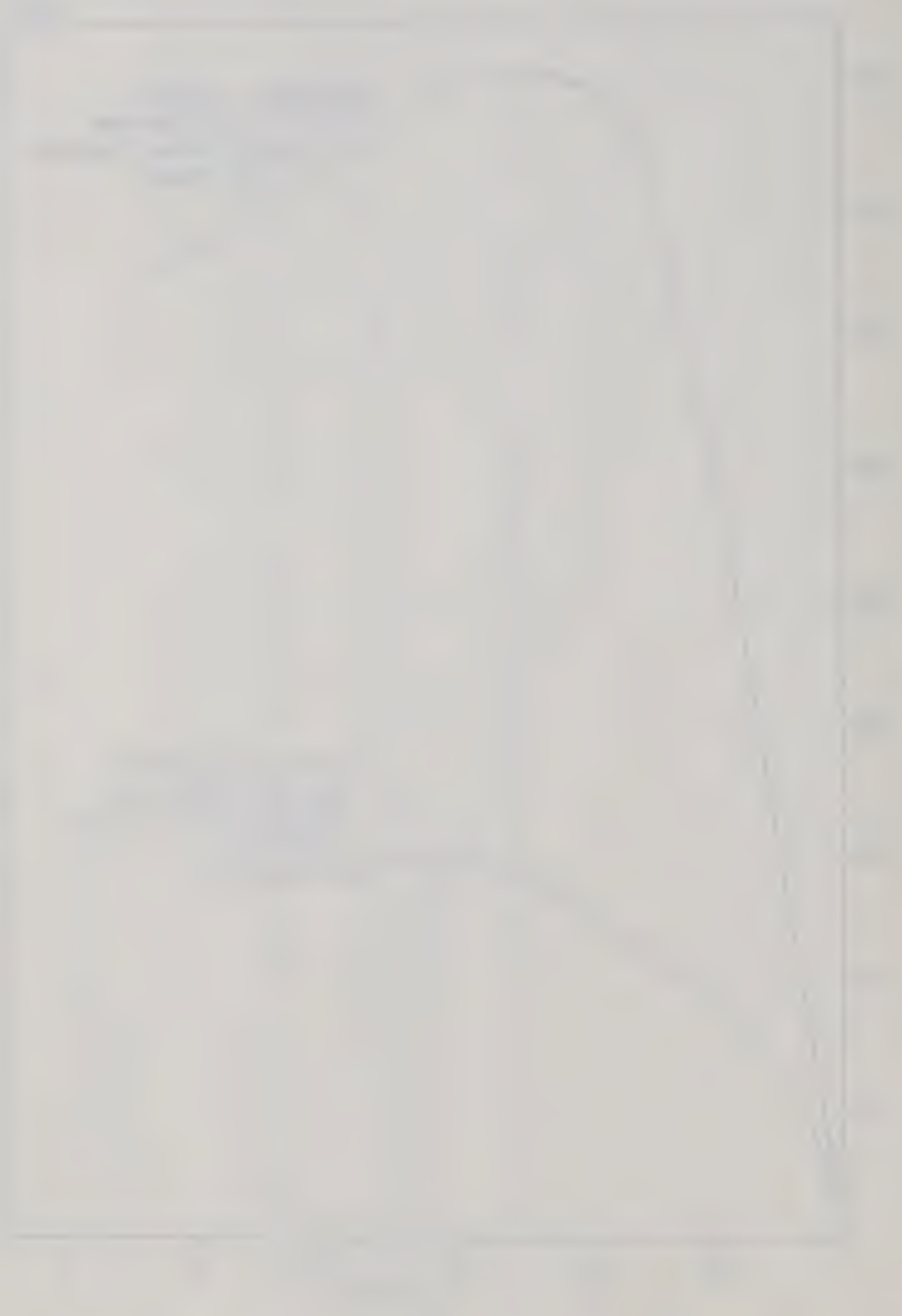


FIG. 4.7. DEVON SLIDE:
EXAMPLES OF STRESS STRAIN CURVES FOR SOFTENED AND UNSOFTENED BACKSLOPE MATERIALS



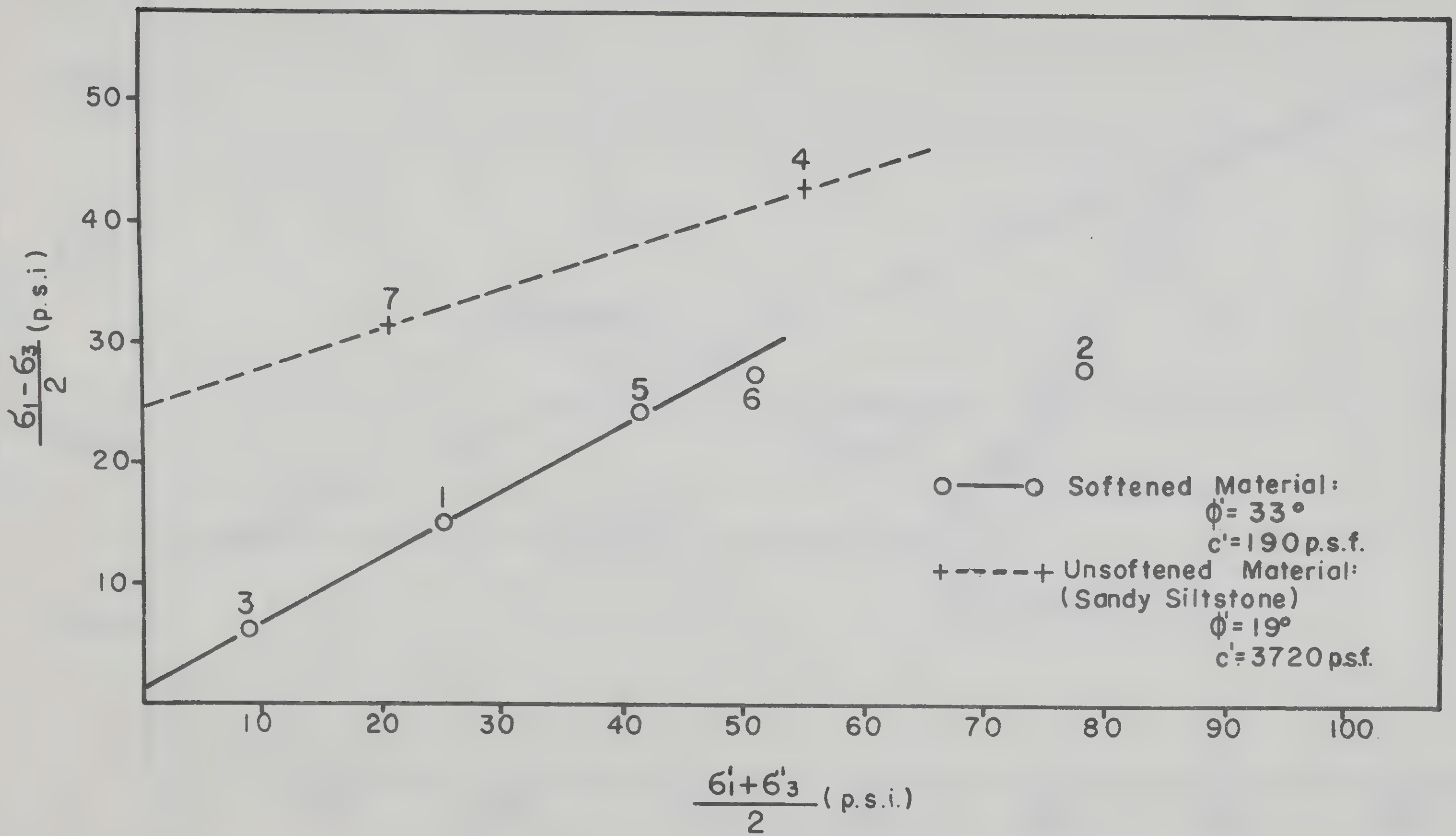


FIG. 4.8. DEVON SLIDE:
RESULTS OF THE TRIAXIAL TESTS ON BACKSLOPE MATERIALS

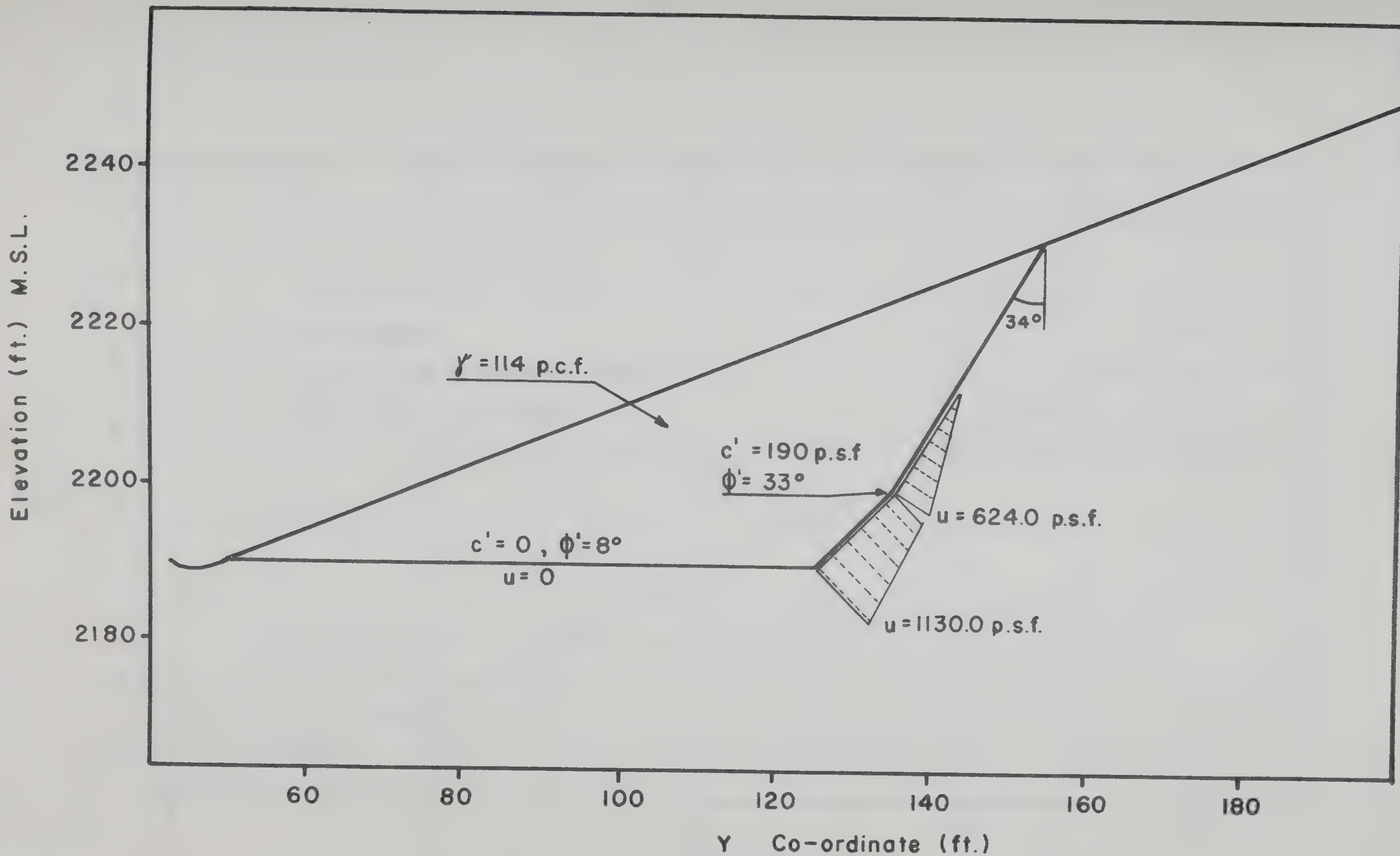


FIG. 4.9. DEVON SLIDE:
FAILURE SURFACE WITH STRENGTH PARAMETERS AND PORE WATER PRESSURES USED FOR THE STABILITY ANALYSIS.

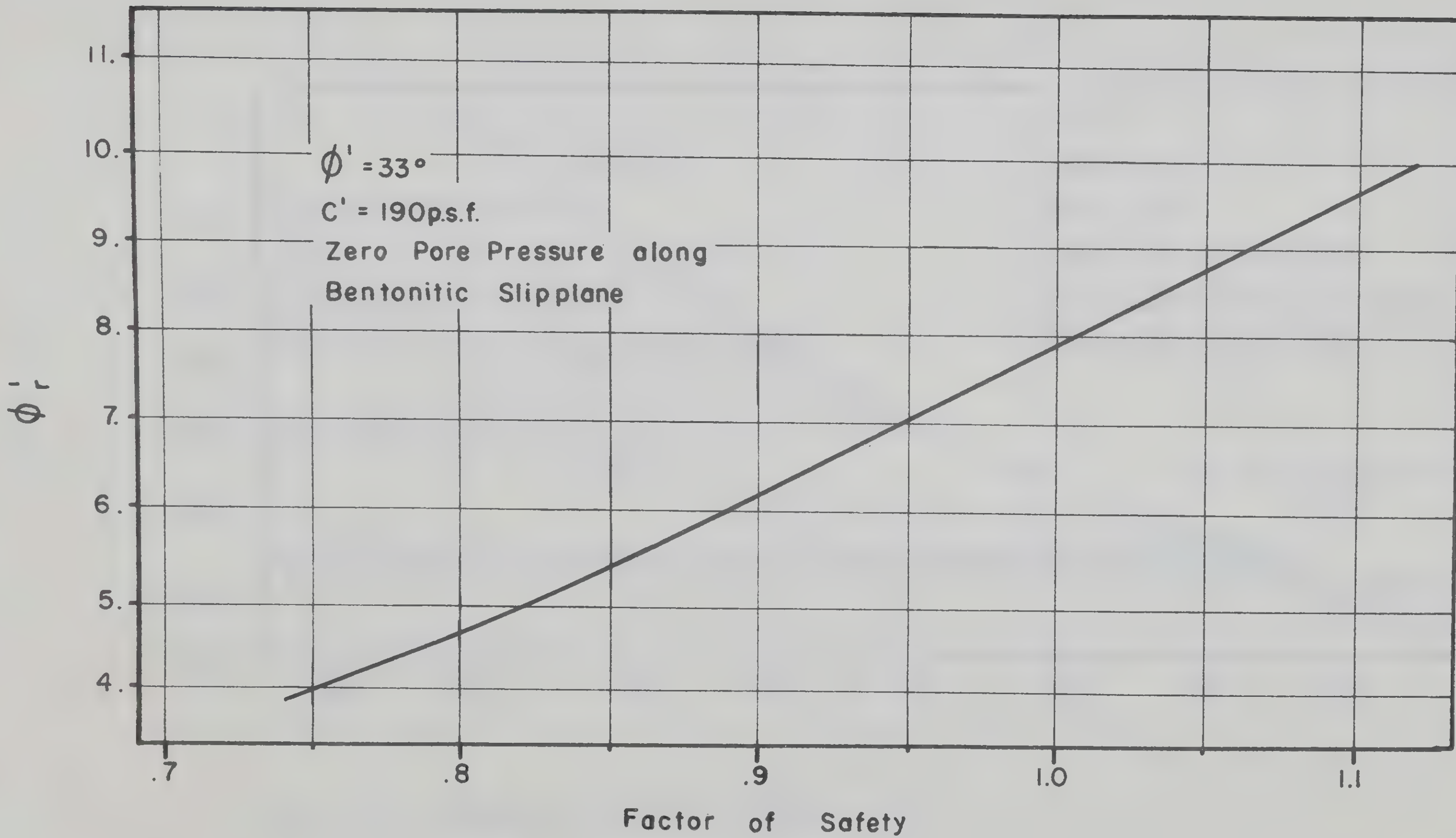


FIG. 4.10. DEVON SLIDE:
VARIATION OF FACTOR OF SAFETY WITH RESIDUAL STRENGTH ALONG BENTONITE

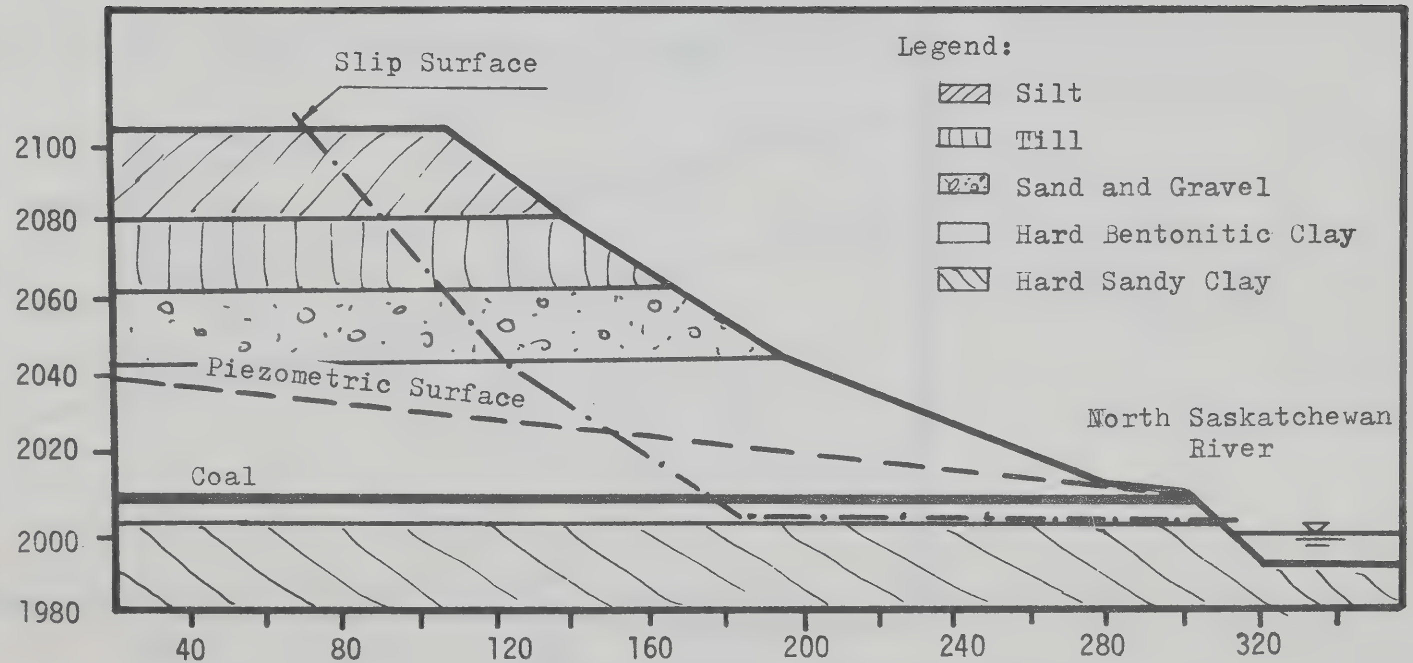
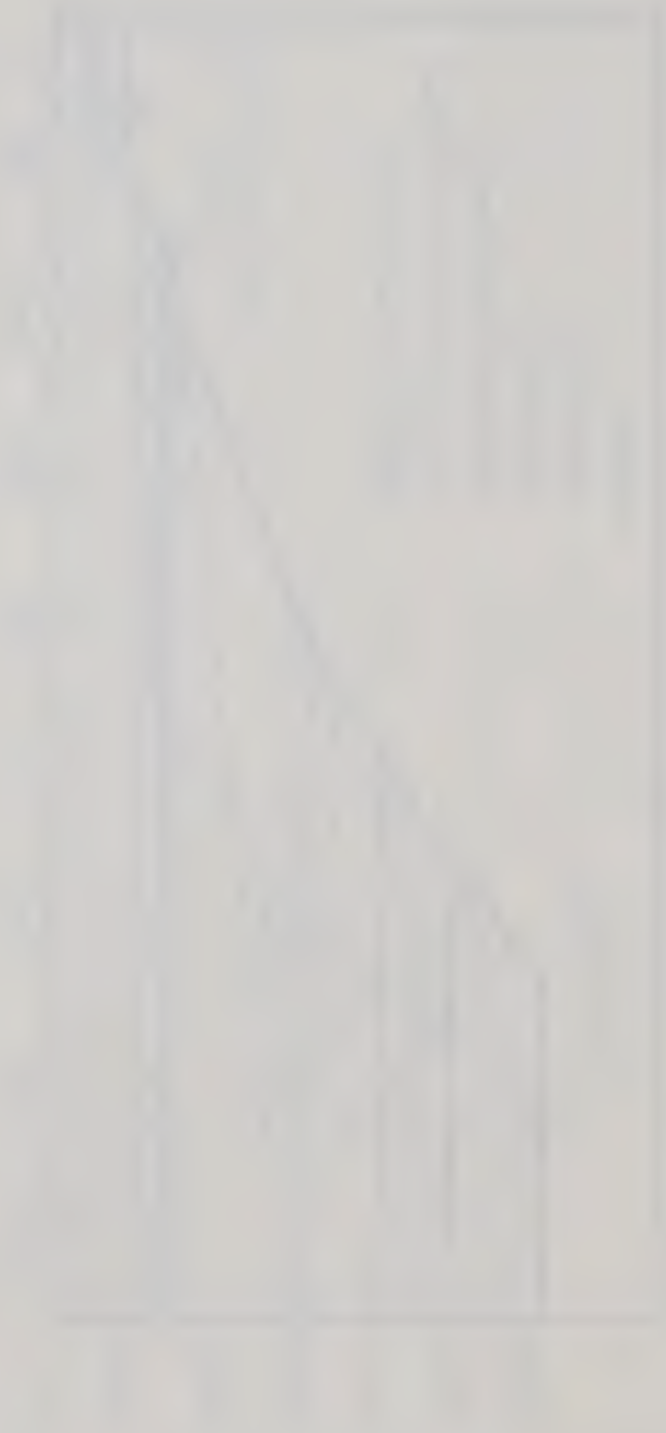


FIG. 4.11. LESUEUR SLIDE (AFTER THOMSON, 1971).



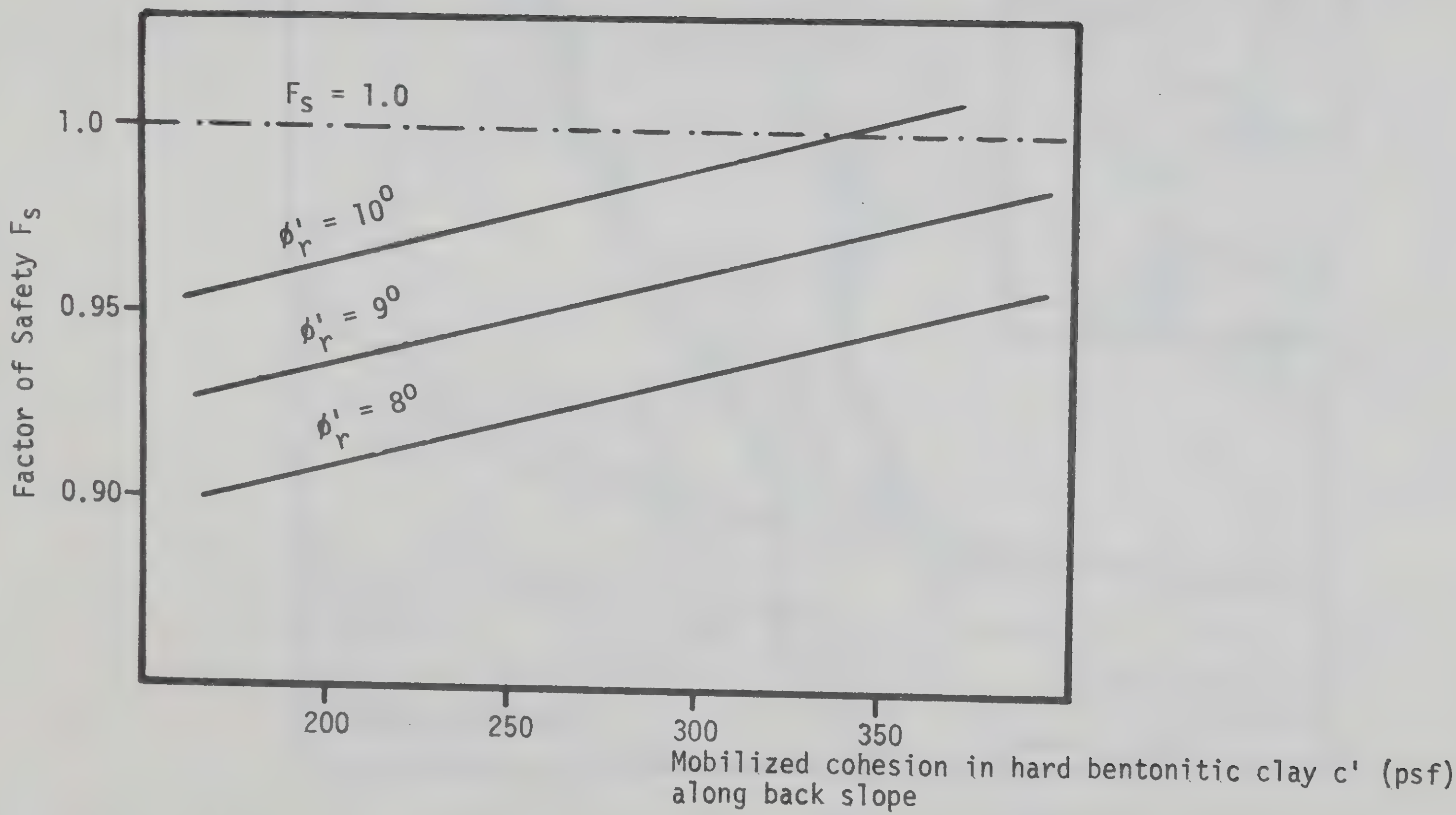


FIG. 4.12. LESUEUR SLIDE: VARIATION OF FACTOR OF SAFETY WITH COHESION MOBILIZED IN HARD BENTONITIC CLAY ALONG BACK SLOPE FOR DIFFERENT FRICTION ANGLES ALONG BEDDING PLANE.

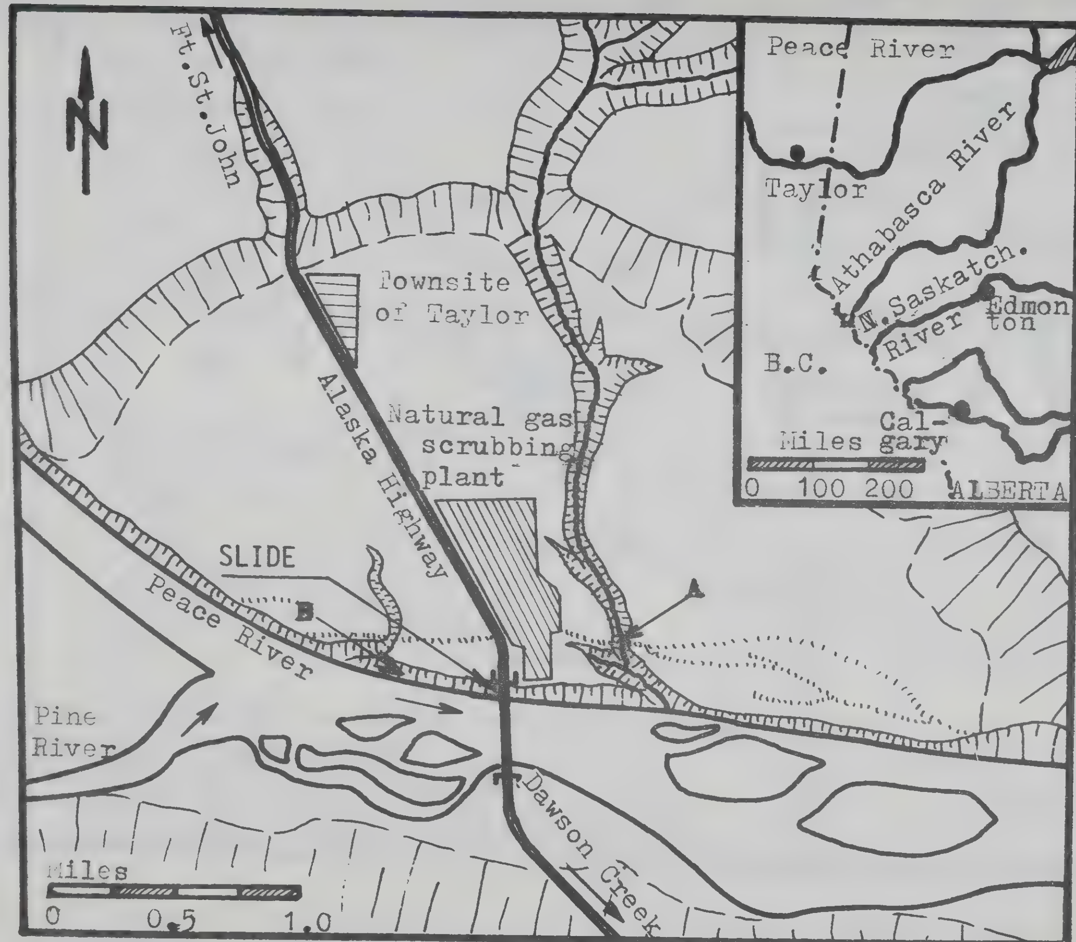


FIG. 4.13. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.: LOCATION OF SLIDE.

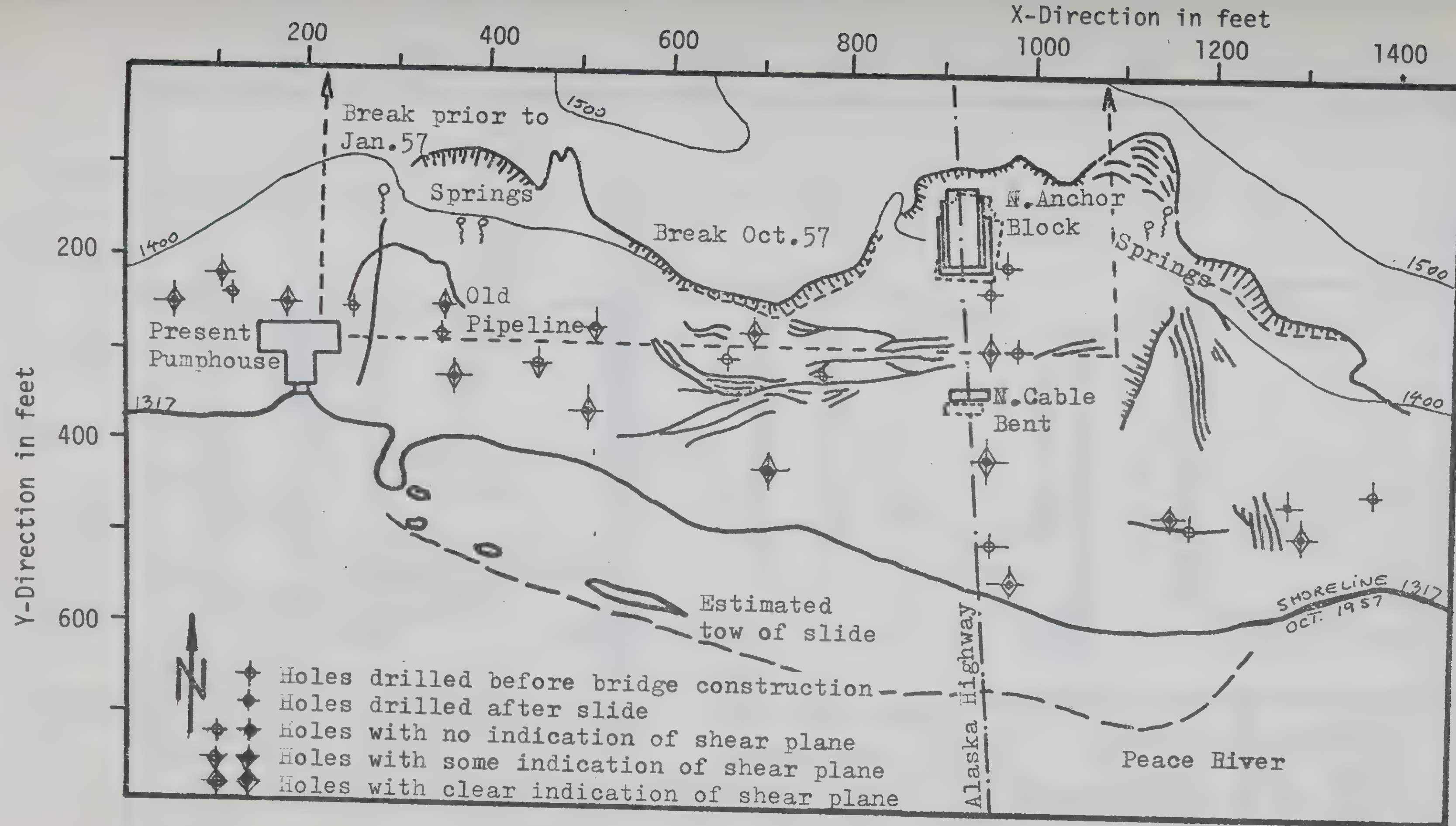


FIG. 4.14. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.: PLAN-VIEW OF SLIDE AREA.

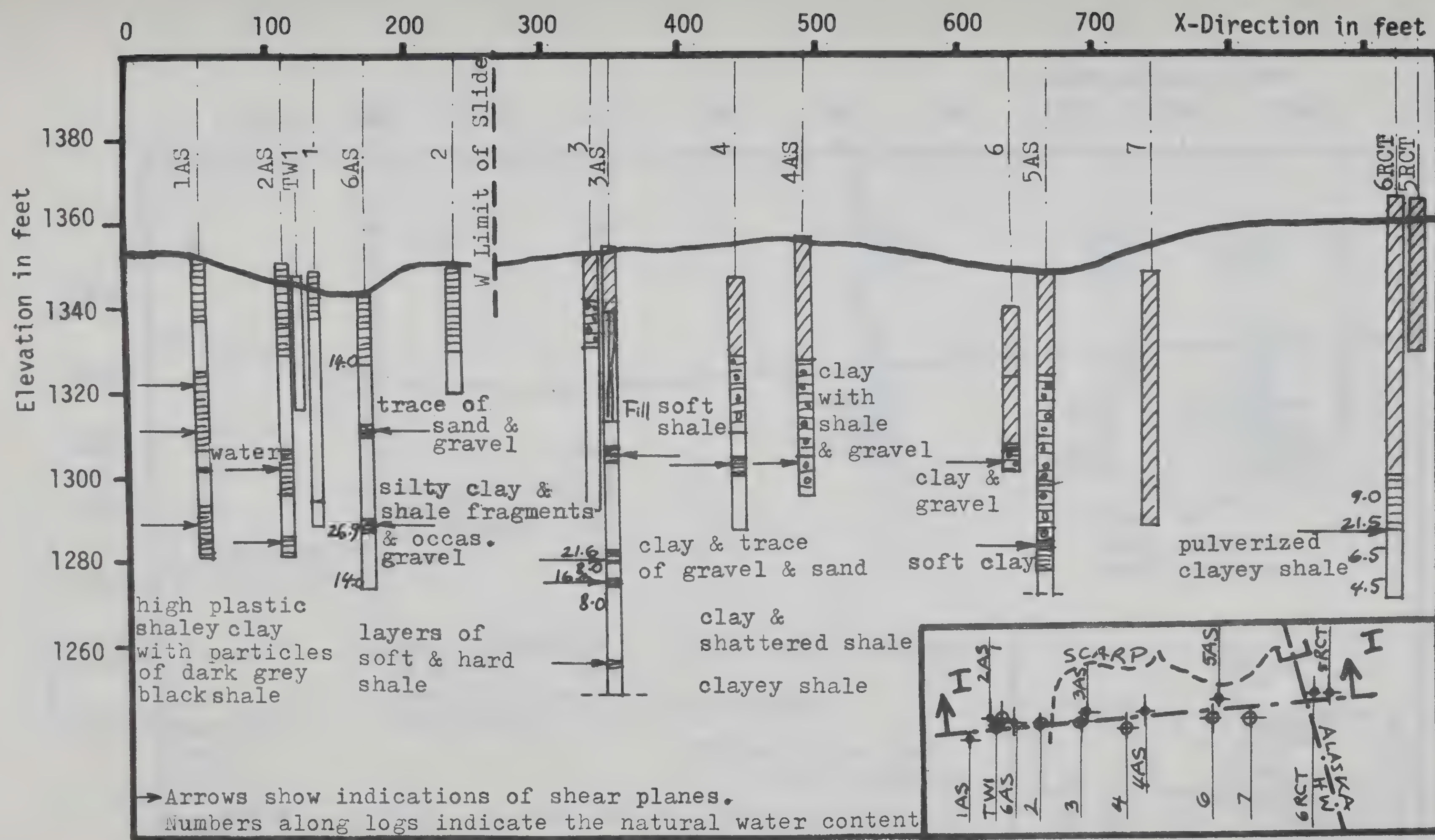


FIG. 4.15. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.:
CROSS SECTION I - I THROUGH SLIDE.

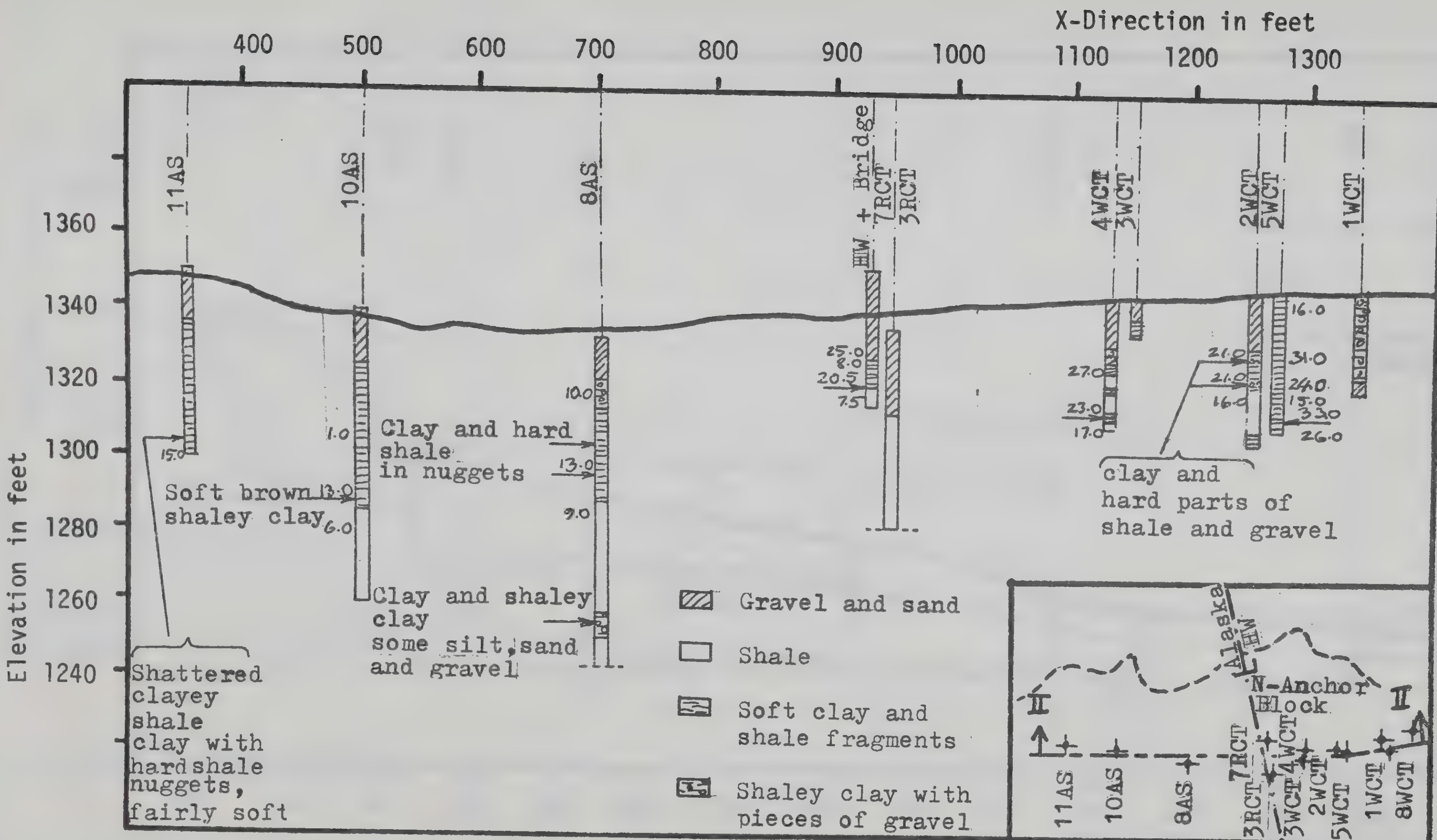


FIG. 4.16. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.: CROSS SECTION II-II THROUGH SLIDE AREA.

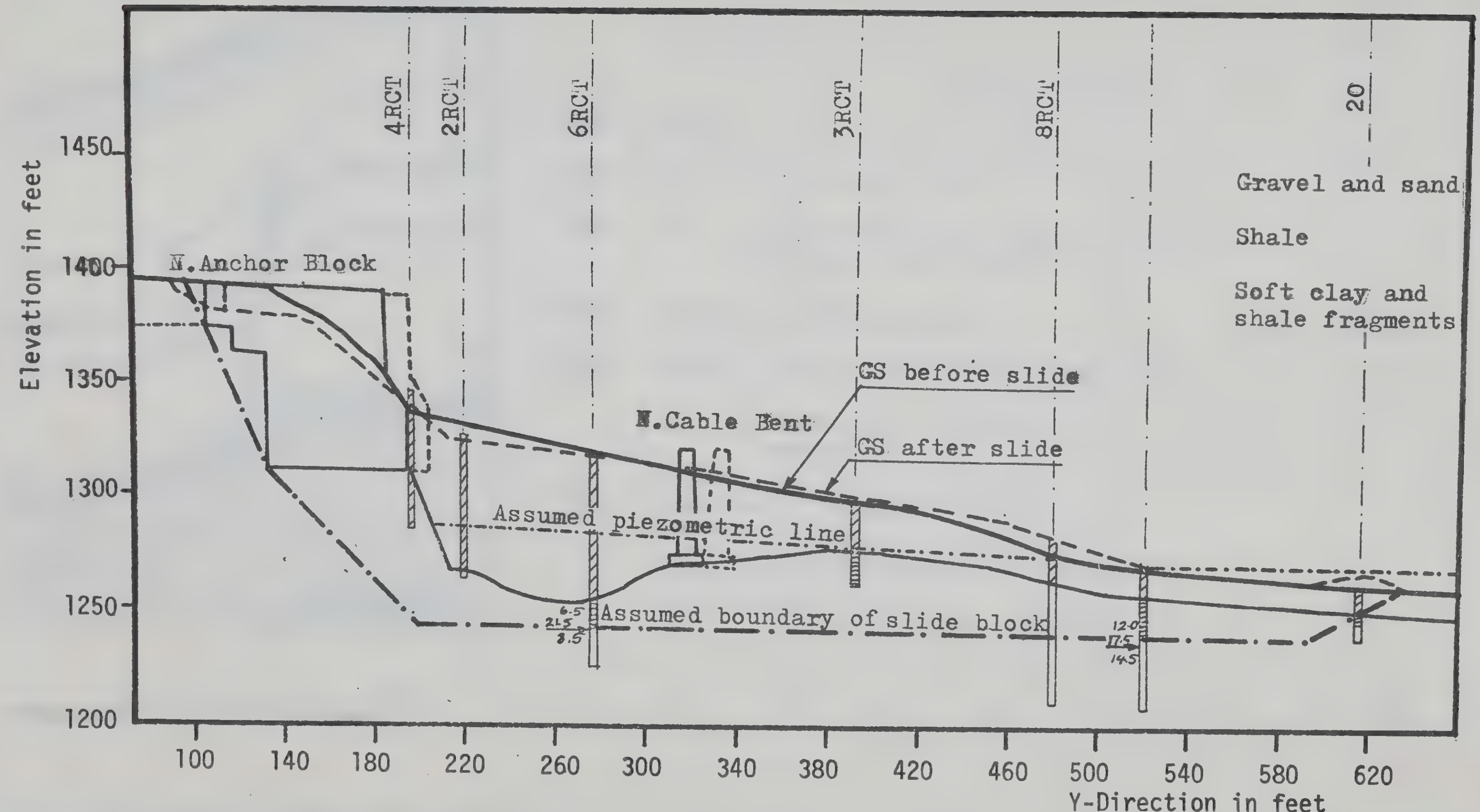


FIG. 4.17. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.: CROSS SECTION III ALONG AXIS OF BRIDGE.

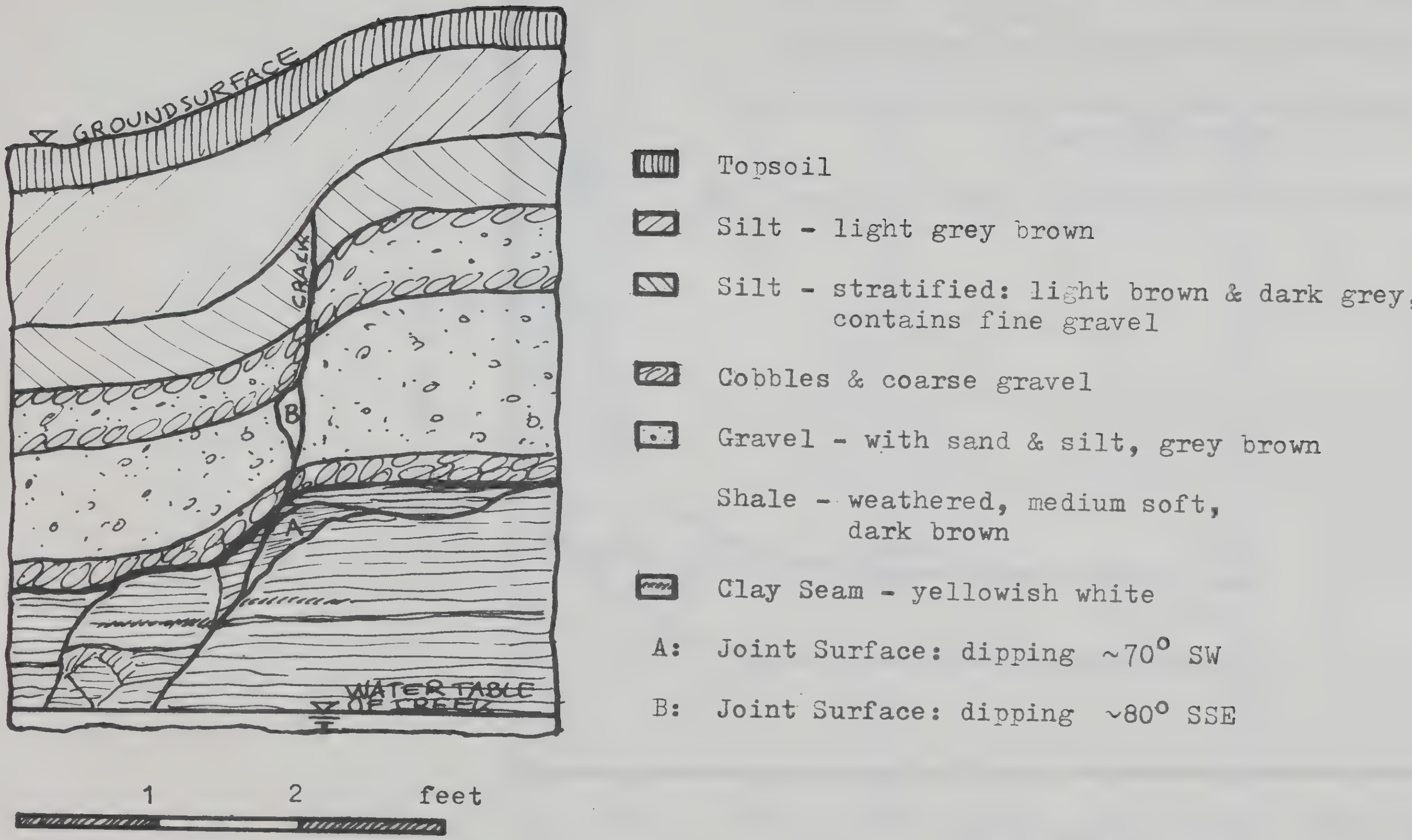


FIG. 4.18. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C.:
PROFILE IN RAVINE OF CREEK EAST OF ALASKA HIGHWAY.

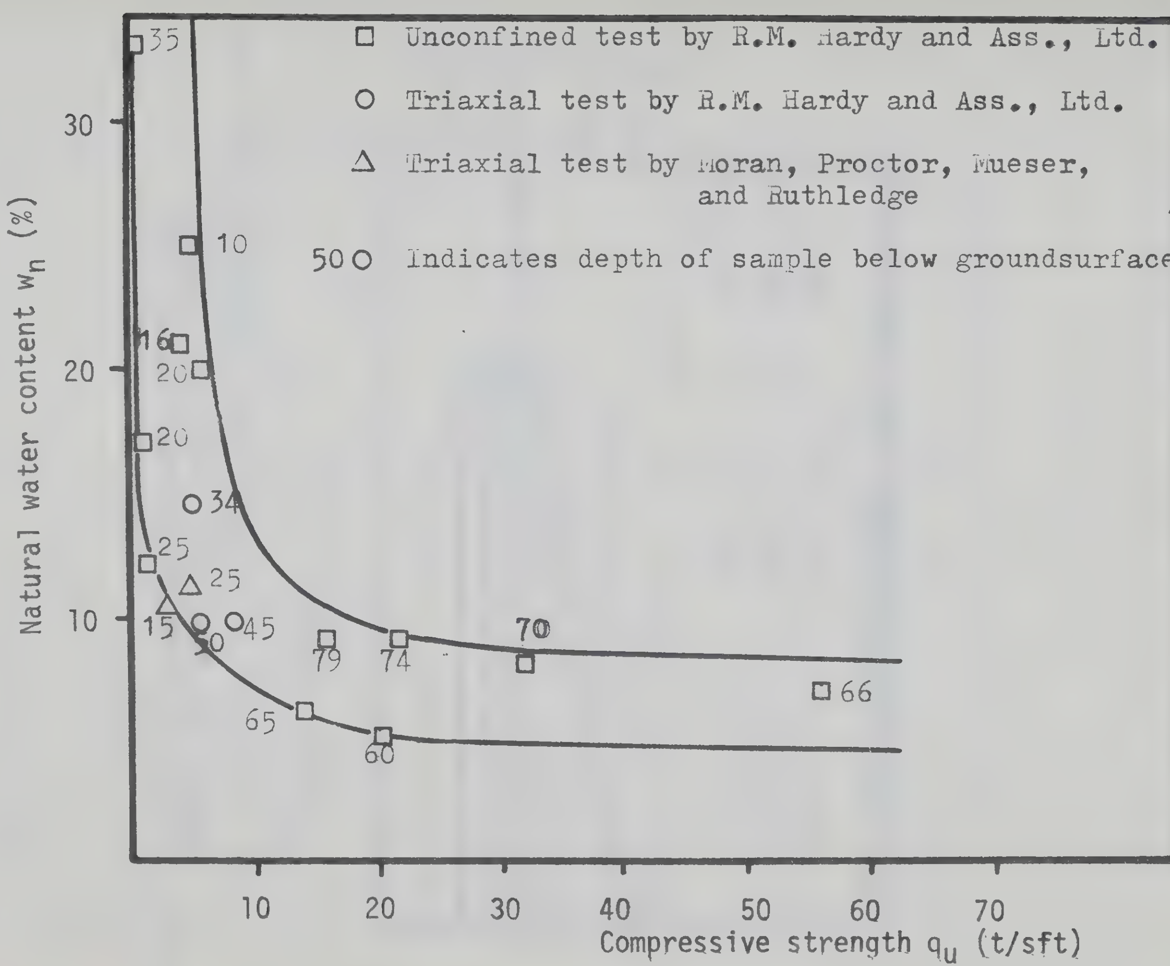


FIG. 4.19. FAILURE OF PEACE RIVER BRIDGE AT TAYLOR, B.C. NATURAL WATER CONTENT VERSUS COMPRESSIVE STRENGTH FOR PEACE RIVER SHALE.

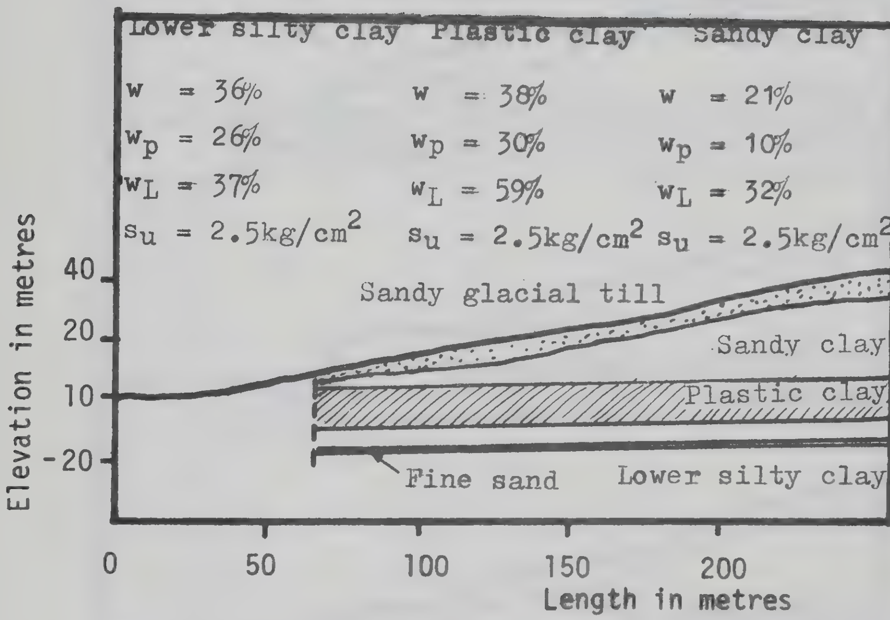


FIG. 4.20. BJERRUM'S CASES:
ORIGINAL GEOLOGICAL CROSS SECTION THROUGH
THE SANDNES SLOPE WITH GEOTECHNICAL DATA OF THE CLAY
(AFTER BJERRUM, 1966).

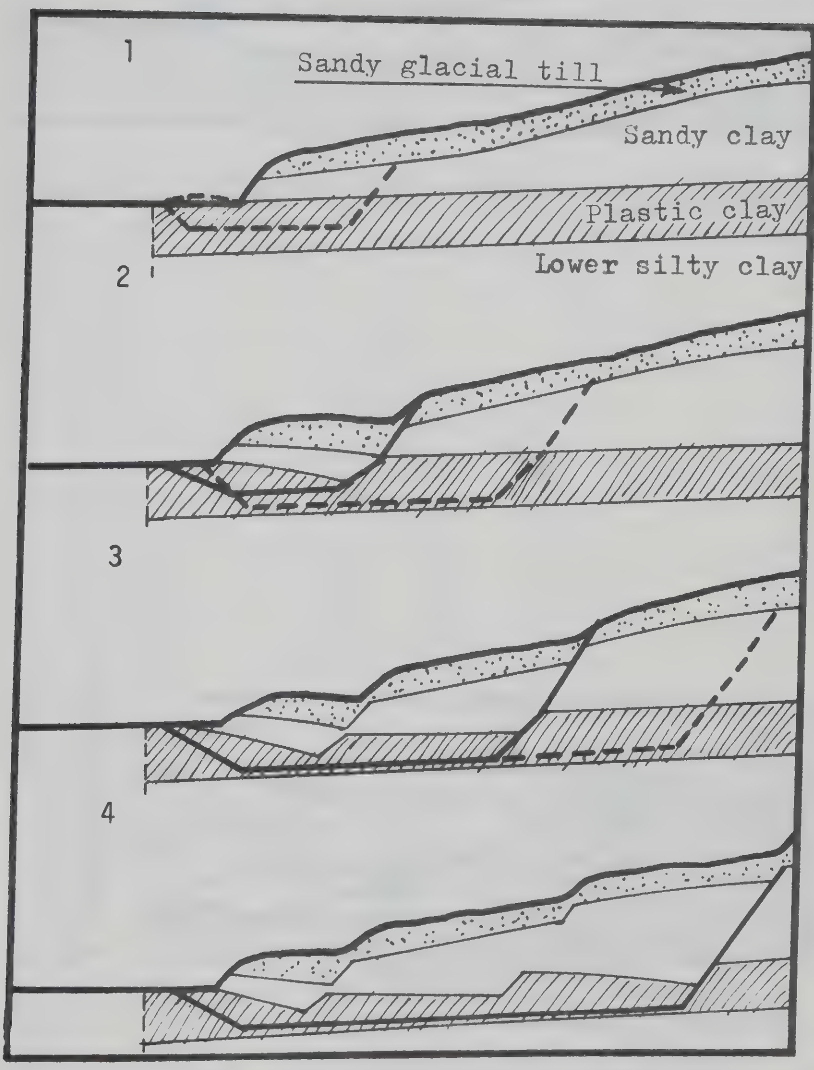


FIG. 4.21. BJERRUM'S CASES:
RECONSTRUCTION OF THE GRADUAL DEVELOPMENT OF FAILURE IN THE SANDNES SLOPE DURING BRICK PRODUCTION (AFTER BJERRUM, 1966).

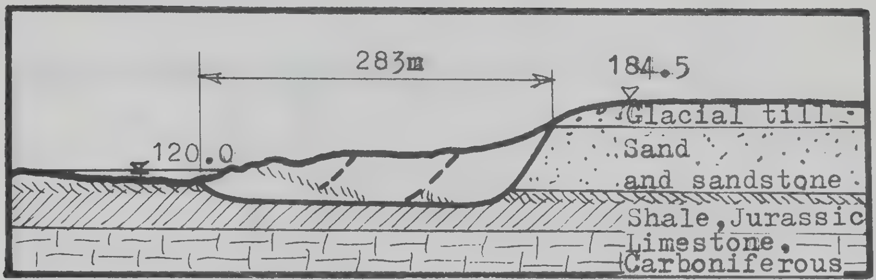


FIG. 4.22. BJERRUM'S CASES: VOLGA SLIDE (AFTER POPOV, 1951).

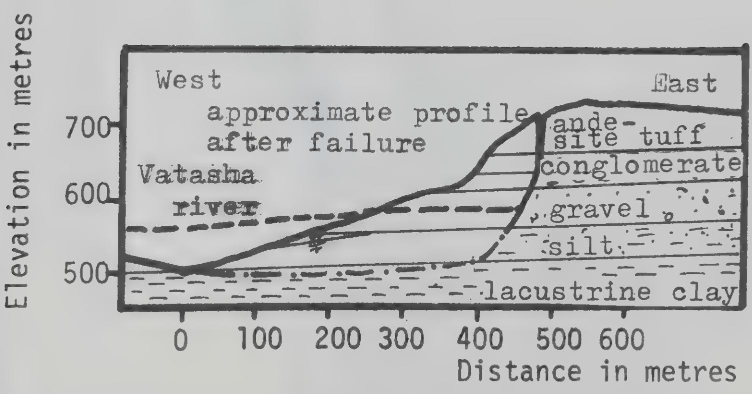


FIG. 4.24. LANDSLIDE AT GRADOT RIDGE, MACEDONIA
(AFTER SUKLJE AND VIDMAR, 1961).

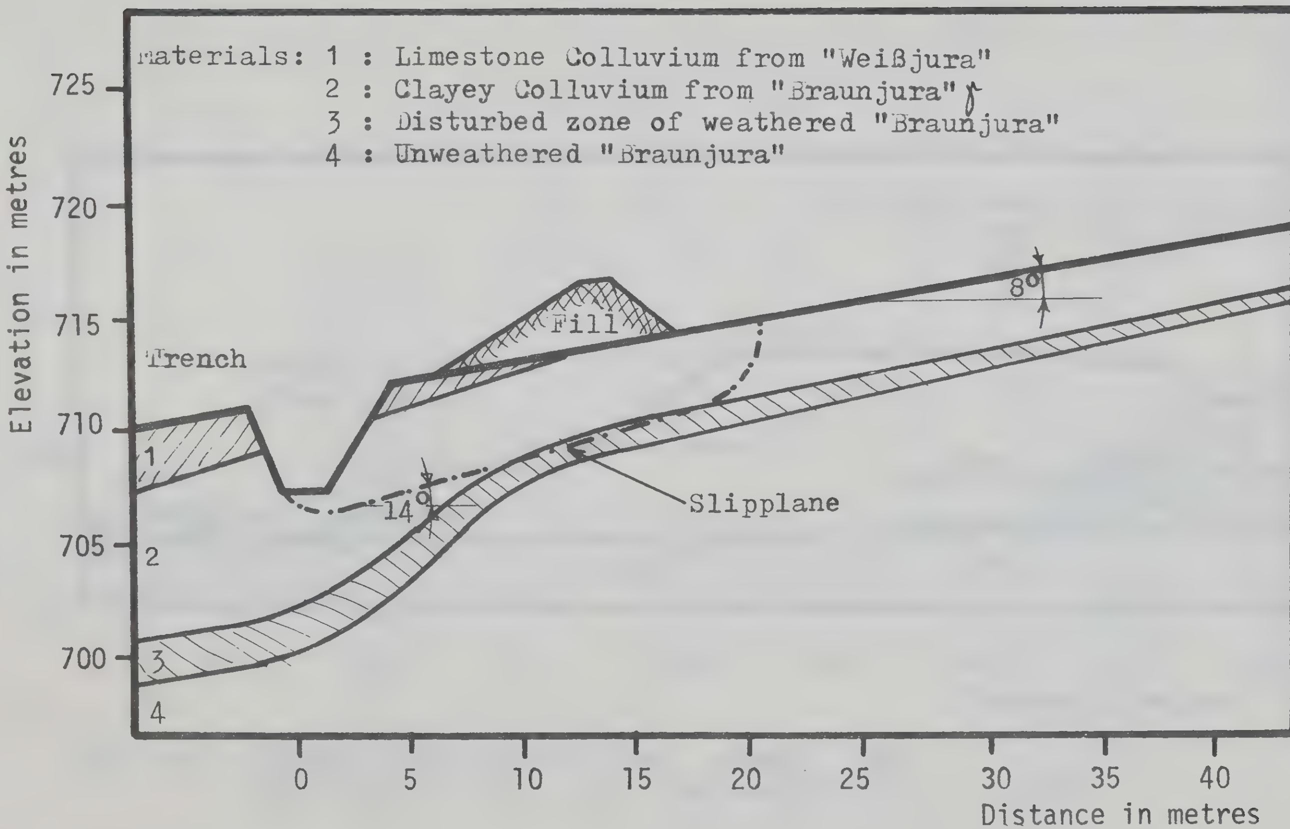


FIG. 4.23. BJERRUM'S CASES:
SLOPE FAILURE IN COLLUVIUM AT BALGHEIM, GERMANY, 1957
(AFTER EINSELE, 1961)

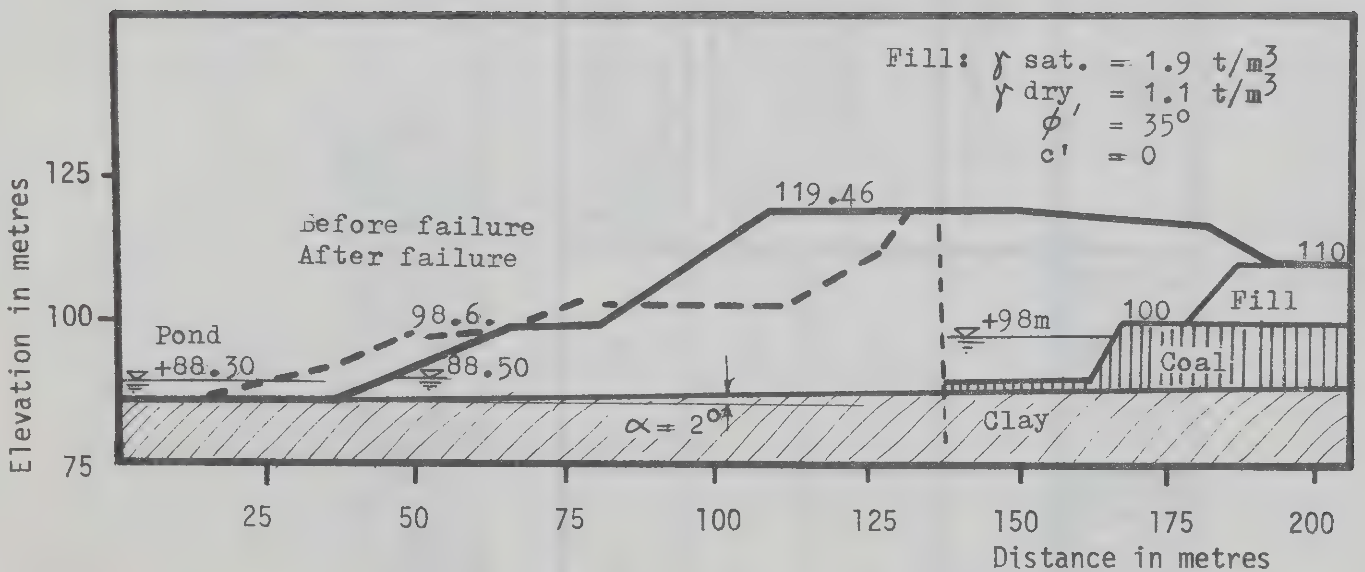


FIG. 4.25. FAILURE IN AN OPEN PIT MINE, GERMANY, 1966
(AFTER BRETH ET AL., 1970, AND WANOSCHEK ET AL., 1970).

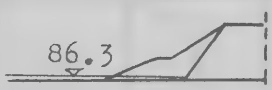
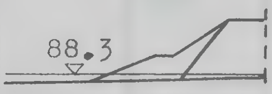
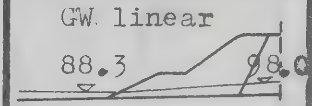
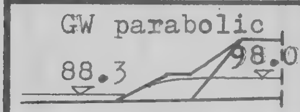
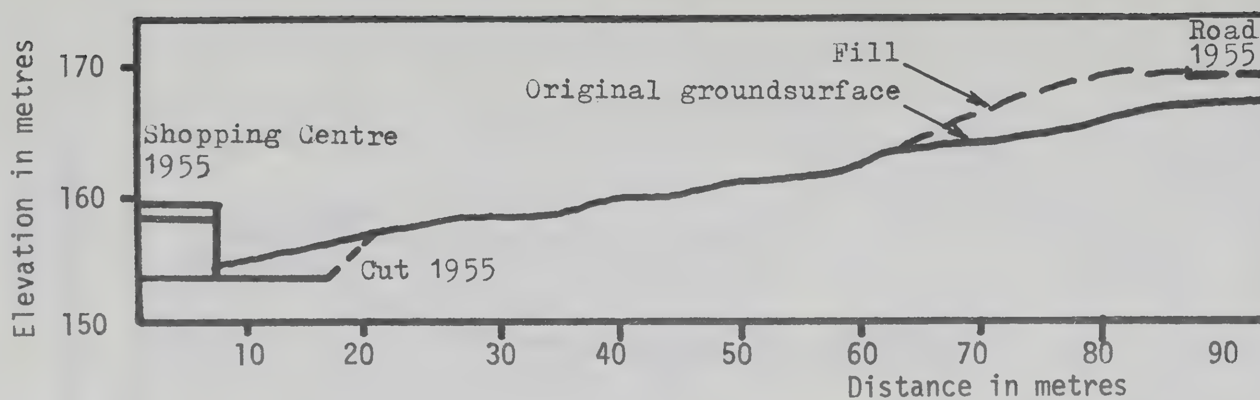
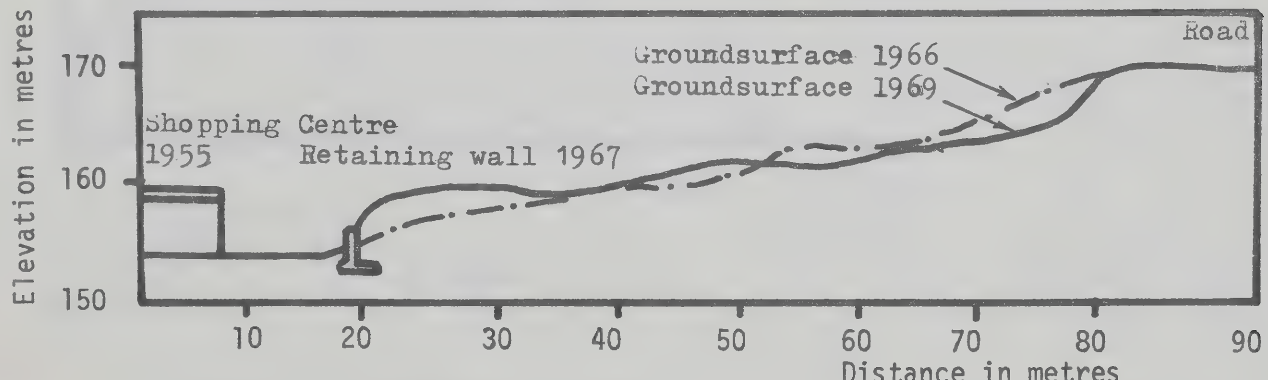
	Backslope: $\phi' = 35^\circ$ $c' = 0$	ϕ' mobilized along base at failure $c' = 0$	difference $\Delta\phi'$ between low W.T. and high W.T.
A		9°	/
B		10°	1°
C		12°	3°
D		16°	7°

FIG. 4.26. FAILURE IN OPEN PIT MINE, GERMANY: INFLUENCE OF GROUNDWATER VARIATIONS ON SLOPE STABILITY ALONG PROFILE A-A (AFTER BRETH AND WANOSCHEK, 1970; WANOSCHEK AND SCHMIDT, 1970).

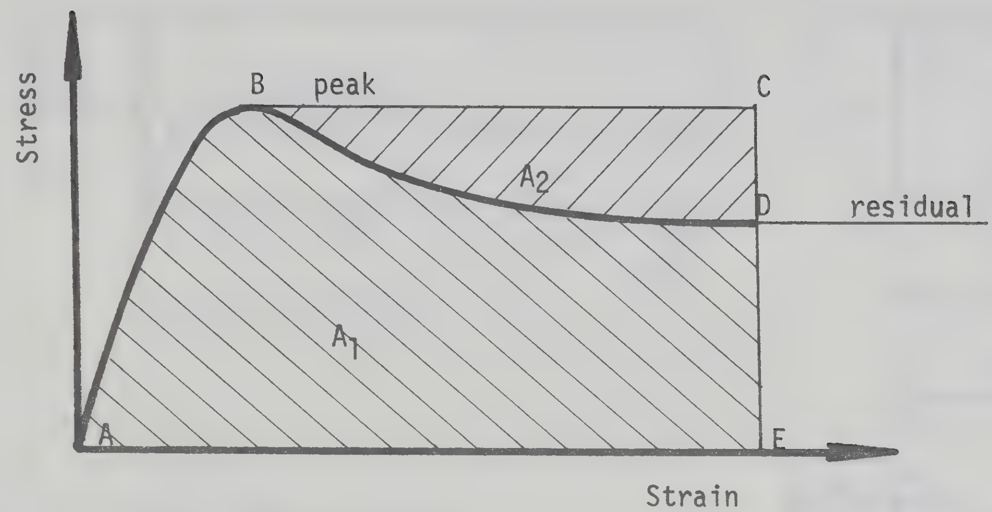


a) Situation before the slide



b) Change of groundsurface between 1966 and 1969

FIG. 4.27. FAILURE IN A SLOPE OF OVERCONSOLIDATED, PLASTIC CLAY, GERMANY, 1961-1969 (AFTER WANOSCHEK AND SCHMIDT, 1970).

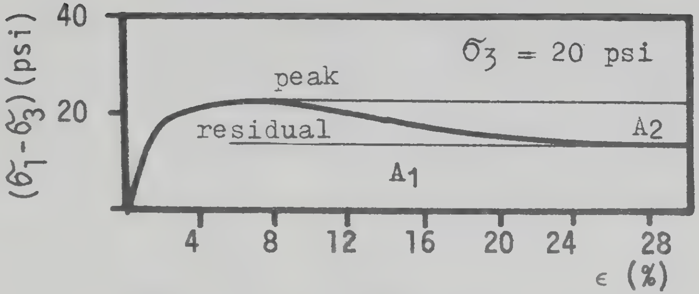


$A_1 \hat{=} A\ B\ C\ E$

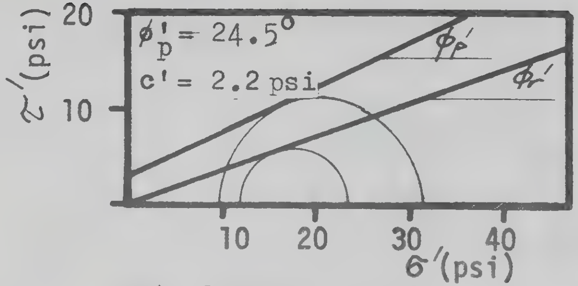
$A_2 \hat{=} B\ C\ D$

$$I_W = \frac{A_2}{A_1}$$

FIG. 4.28. DEFINITION OF THE WORK SOFTENING INDEX I_W.



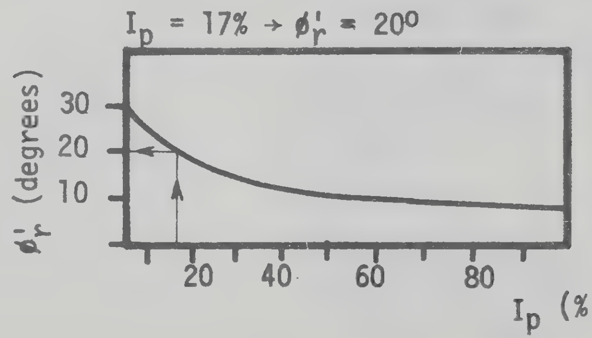
Stress versus Strain



Mohr Diagram

Work Softening Index:

$$I_W = \frac{A_2}{A_1} = \frac{105}{590} = 0.177$$



Residual angle of friction versus Plasticity Index (Bjerrum, 1967)

FIG. 4.29. EXAMPLE FOR THE DETERMINATION OF THE WORK SOFTENING INDEX FROM CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS ON WELLAND CLAY (KWAN, 1971).

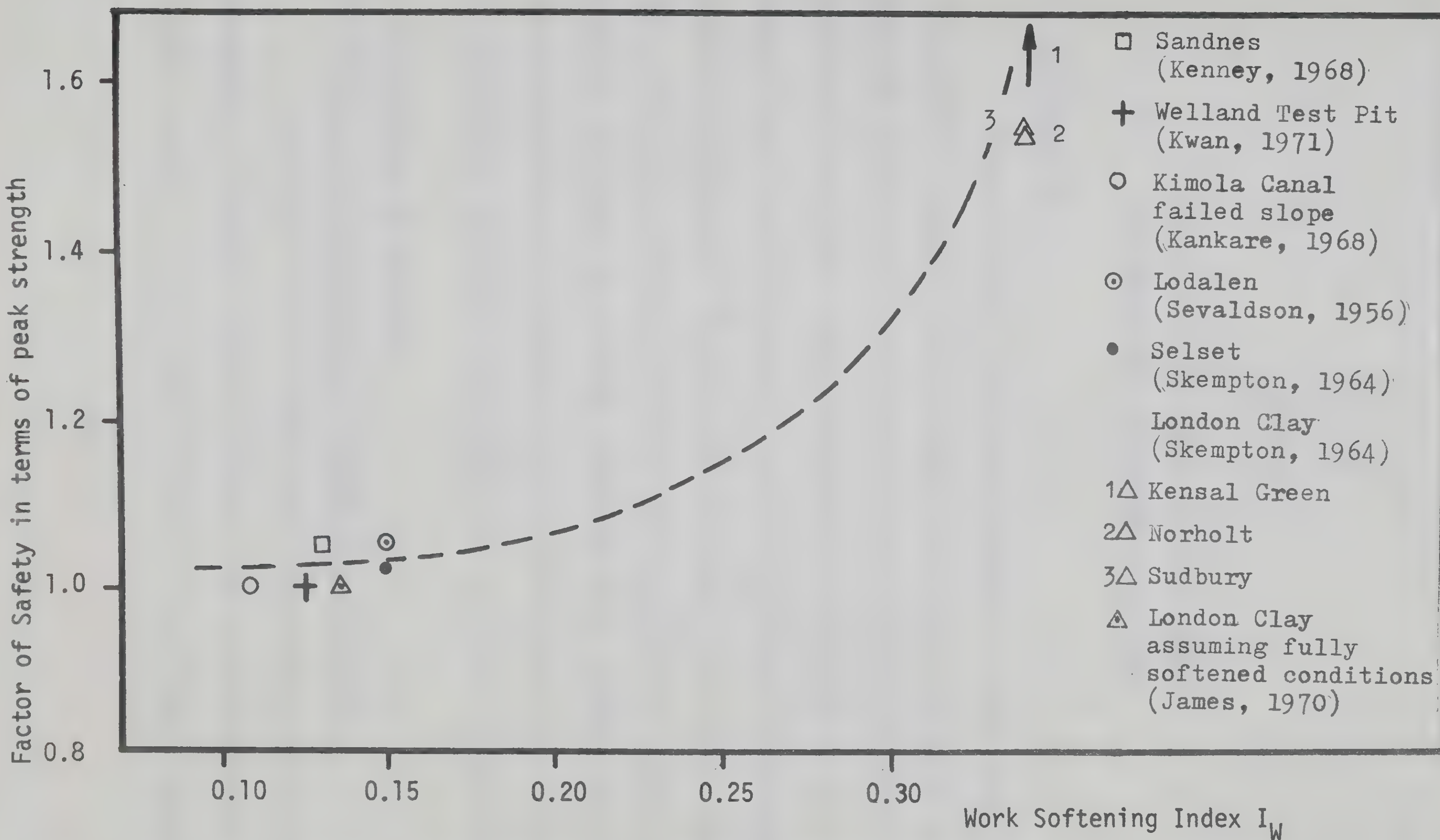


FIG. 4.30. FACTOR OF SAFETY IN TERMS OF PEAK STRENGTH VERSUS WORK SOFTENING INDEX.

CHAPTER V

ANALYSIS OF THE PORE PRESSURE CHANGES FOLLOWING THE EXCAVATION OF A SLOPE

1. Introduction

Excavation of a cut causes unloading of the ground, which consequently expands under undrained conditions, and leads to pore pressure reductions as described by Bishop and Henkel (1953). The pore pressures immediately after excavation tend to equalize until steady seepage conditions are reached. During this process of dissipation of excess pore pressures the effective stresses in the ground change. Since the excess pore pressures due to unloading are mainly negative with respect to the final equilibrium conditions they increase and hence cause the average effective stresses in the slope to decrease. This reduction of the average effective stress may lead to failure of the slope.

2. Description of Analyses

2.1. Analysis of Pore Pressure Changes due to Unloading

The total stress changes due to excavation of a slope in a homogeneous, isotropic, elastic material may be calculated by a Finite Element Analysis. In the present study a Finite Element program was used

which was written originally by Wilson (1963) for plane stress or strain analyses of solid mechanics problems. It has been modified for automatic generation of nodes and elements and for the calculation of pore pressure changes according to Skempton(1954). Constant strain triangles are used in the program. The unloading effect is simulated by applying stresses to the excavated surfaces which are equal in magnitude but opposite in sign to the stresses in the ground before excavation (Duncan and Dunlop, 1969). The unloading stresses upon excavation are shown in Fig. 5.1. Assuming a water table equal to the original groundsurface and a hydrostatic pore pressure distribution the unloading stresses are:

$$\sigma_v = \gamma_t \times h$$

and

$$\begin{aligned} \sigma_h &= K_o \sigma_v' + u \\ &= K_o(\sigma_v - u) + u \\ &= K_o h\ (\gamma_t - \gamma_w) + h\gamma_w \\ &= h\ [K_o(\gamma_t - \gamma_w) + \gamma_w] \end{aligned}$$

In which: σ_v = vertical total stresses

σ_h = horizontal total stresses

γ_t = total unit weight

γ_w = unit weight of water

K_o = Coefficient of Earth Pressure at Rest

u = pore pressure

h = distance below original groundsurface

In order to minimize the influences of the boundaries, the dimensions and boundary conditions of the Finite Element model of the slope are chosen according to the suggestions by Duncan, Dunlop, and Seed (1968). In Fig. 5.2. the model is shown which has been used for the analysis of a slope with an inclination of two horizontal to one vertical (2 to 1). The width D_B is taken smaller than suggested, assuming that the excavated pit has the width $W = 2D_B$. In order to increase the accuracy of the results for the slope zone a finer grid is chosen for the upper regions than for the regions below the base of the excavation. The magnitude of the Modulus of Elasticity is of no importance for the unloading process as long as a homogeneous, isotropic, elastic material is considered, and the magnitude of deformations is of no interest. Values of Poisson's Ratio ν have been reported to range between 0.1 and 0.5 (Jaeger, 1964). If a soil is assumed incompressible ν is 1/2. For the present analysis a value of $\nu = 0.3$ is chosen, even though saturated soils under undrained conditions should be considered incompressible. The influence of ν, however, on the calculation of stress changes under plane strain conditions is not very strong.

The stress changes are calculated by applying the loads in one single step (Dunlop, Duncan, and Seed, 1968). The changes in pore pressures due to the stress changes in a triaxial test were formulated by Skempton (1954) as:

$$\Delta u = B\ [\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\ ]$$

in which: Δu = changes in pore pressures

$\Delta\sigma_1$, $\Delta\sigma_3$ = major and minor changes in total stresses

A, B = Pore Pressure Coefficients

For saturated soils the pore pressure parameter B is equal to one, therefore

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$$

The pore pressure parameter A varies for any given soil with the stresses and strains. Skempton(1954) reported A-values at failure due to positive stress increments in the range of A = 1.5 for high sensitive clays and A = (-1/2 to 0) for heavily overconsolidated clays. Values of A = 0.7 were reported for slightly overconsolidated medium plastic clays from Welland (Kwan, 1971) and the Kimola Canal (Kankare, 1968). In both cases the values were obtained from triaxial tests under loading conditions. For an ideal elastic soil A is 1/3 during triaxial loading conditions, or A is 1/2 during plane strain conditions.

The present study is concerned with the unloading under plane strain conditions of slopes in overconsolidated clays, which have not failed during this process. For the analysis A-values are chosen which range from zero to 0.5. The principal stresses are considered to be acting on vertical and horizontal planes before excavation. Due to unloading the principle stresses rotate. The rotation, however, has been

found in general not to be very substantial, and therefore, as an approximation, the stress changes in the x- and y- directions can be considered identical to the changes of the principal stresses:

$$\Delta\sigma_x \simeq \Delta\sigma_1$$

$$\Delta\sigma_y \simeq \Delta\sigma_3$$

Thus $$\Delta u = \Delta\sigma_y + A(\Delta\sigma_x - \Delta\sigma_y)$$

The pore pressures immediately after excavation u_o are determined from the difference between the pore pressures before excavation u^* and the pore pressure changes due to unloading Δu:

$$u_o = u^* - \Delta u$$

The pore pressures u_o are calculated by assuming a hydrostatic pore water distribution in the ground before excavation:

$$u^* = \gamma_w \times h$$

The difference in pore pressures between the condition immediately after excavation u_o and the stabilized groundwater conditions u_t represent the excess pore pressures Δu_e which tend to dissipate to reach equilibrium groundwater conditions:

$$\Delta u_e = u_o - u_t$$

2.2. Analysis of the Dissipation of Excess Pore Pressures

The time for equalization of the excess pore pressures within a slope can be calculated by using Koppula's two-dimensional consolidation program (Koppula, 1971), assuming that consolidation and swelling follow the same theory. The equation governing the two-dimensional dissipation of excess pore pressure under plane strain conditions, written in a dimensionless form, is:

$$\frac{\partial^2 u_e}{\partial X^2} + \frac{\partial^2 u_e}{\partial Y^2} = \frac{\partial u_e}{\partial \tau}$$

in which: u_e denotes excess pore pressure at a point (x,y)

τ denotes the time factor ($= c_v t/H^2$)

c_v denotes the coefficient of consolidation of the embankment material

t denotes the time since the inception of dissipation

H denotes the height of the embankment

X denotes x/H

Y denotes y/H

This equation is called the Terzaghi Equation for consolidation in two dimensions. The equation is a special case of the three-dimensional diffusion of water through a porous medium, which has been developed from the simple consolidation theory by Terzaghi (1923) and Rendulic (1936). The equation is based exactly on the same assumption as the one-dimensional theory. The equation was solved by Koppula (1970) by using a Finite Difference approximation. The computer program set up

for this purpose solves the consolidation characteristics of an embankment for the boundary conditions as shown in Fig. 5.3. No drainage is specified along the base of the slope and along a vertical boundary which is considered far away from the excavated face. The program reads in all the input data, regarding the material properties and the geometric properties of the embankment. The original program is modified so that the initial excess pore pressures can also be read. Depending on the governing material and operating parameters the program calculates the new excess pore pressures at prescribed time factor intervals. For the almost complete equalization of the excess pore pressures throughout the cross-sectional area the time factor for full dissipation can be obtained and the time for equalization of pore pressures can be calculated. It should be remembered that 100% equalization is reached after infinite time only. Nevertheless, the time period, after which no more excess pore pressures are detectable, will be called the time for full dissipation of excess pore pressures.

3. Presentation of Results and Comparison with Field Observations

3.1. General

In order to evaluate the usefulness of the analysis, it has to be applied to actual cases and the results have to be compared with field observations. However, only very few complete field data exist at present of pore pressures immediately after excavation of a slope and the consequent pore pressure equalization. Soil parameters such as

K_o, A, c_s, are only vaguely known for field conditions, so that even a very detailed analytical investigation would provide only very approximate results. Considering both, scarceness of comparable field data, and little information of governing soil parameters, the analysis can be only a preliminary study at the present stage.

The pore pressure change due to unloading during excavation of a 100 feet high slope with the inclination of two horizontal to one vertical (2 to 1) is analysed first. The Pore Pressure Coefficient A for a highly overconsolidated clay has been assumed to be A = 0.3 and A = 0. K_o-values of 1.0 and 1.5 are considered for the determination of the horizontal unloading stresses. The Finite Element model of the slope is indicated in Fig. 5.2. The computed pore water distribution immediately after excavation is shown in Figs. 5.4., 5.5., and 5.6. It can be recognized that the pattern of pore pressures within the slope region does not vary substantially for different assumptions of K_o- and A- values. The assumed steady seepage conditions are represented by a flow net as indicated in Fig. 5.7.

The excess pore pressures from the analysis for K_o = 1.5 and A = 0.3 have been read into the two-dimensional consolidation program. The dissipation of pore pressures has been calculated for time-step intervals of τ= 0.01. For almost complete equalization of pore pressures a time factor τ= 0.33 has been obtained. Very small excess

pore pressures are reached already around $\tau = 0.20$. The pore pressure distribution after the first time-step is shown in Fig. 5.8. A comparison with the initial pore pressure distribution obtained for $K_o = 1.0$ and $A = 0.3$ indicates similar values. This shows that variations in initial pore pressures due to different assumptions of K_o- and A-values are equalized at very early stages of dissipation, and therefore are little reflected during the later stages of pore pressure equalization.

By knowing the time factor for full pore pressure equalization $\tau = c_v t/H^2$, the influence of slope height H on time for dissipation t can be evaluated, as well as the influence of Coefficient of Swelling c_s. In Fig. 5.9. c_s is plotted against t for constant embankment heights H and in Fig. 5.10. H is plotted against t for constant Coefficients of Swelling c_s. Both plots indicate that the time for almost complete equalization of pore pressures increases substantially with increasing slope height in materials with low c_s-values but only slightly for materials with high c_s-values. The results obtained so far from the pore pressure analysis will be compared with field data in the following.

3.2. Comparison with Field Observations

Slope at Kimola Canal, Finland (Kankare, 1969)

The Kimola Canal is situated in Central Finland and was built between the winter of 1962 and the summer of 1966. The slopes in highly to slightly overconsolidated clays had been designed on the basis of

a $\phi = 0$ - Analysis for a Factor of Safety $F_s = 1.5$. In 1963 a large number of failures occurred which threw serious doubts on the $\phi = 0$ - method as a design basis for this canal. A detailed investigation was started which included common weight soundings, vane borings, effective strength tests, and very detailed pore pressure measurements.

The main part of the additional investigations, including pore pressure measurements, had been carried out at station 52+70. During a large slide in November 1965 all the installations were destroyed. Therefore, after this slide the field investigations were continued by measuring pore pressures at the upper canal at station 43+00, where the slopes, so far, are still intact. The properties of the soil at the lower canal at station 52+70 are summarized in Table V.1.

Continuous pore pressure measurements were obtained at station 57+70 from July 1964 until November 3rd, 1965, when the failure occurred and at station 43+00 from spring 1966 to spring 1968. In all cases open standpipe piezometers of the Geonor type were used. The piezometers were reported as operating faultlessly on the whole. The time lag of the piezometer instrumentation was found to be very short. Balanced readings could be recorded within a few hours. During the excavation of the canal at station 52+70 between January 24th and February 2nd, 1965, the piezometers recorded a rapid drop of pore pressures. The lowest values were observed on February 1st and 2nd when the exca-

vation was finished and water was again rising in the canal between the cofferdams. Equalization of pore pressures was reached approximately 6 months later. The pore pressures reported for April 20th, 1965, indicate that steady seapage conditions were not reached at this stage. However, the pore pressures obtained on July 25th, 1965, indicate similar seepage conditions as measured at later periods (See Fig. 5.11.).

The pore pressures upon unloading and the consequent pore pressure dissipation are analysed for this slope section assuming $K_0 = 1.0$ and $A = 0.3$. The computed change of pore pressures due to excavation agrees surprisingly well with the measured values. A comparison between field results and analytical results is given in Fig. 5.12. The analysed time for full pore pressure equalization is 1.5 years for $c_s = 30$ ft^2/year, or 0.7 years for $c_s = 60$ ft^2/year, which is also the same order of magnitude as observed in the field. Pore water pressures are computed for intermediate time stages, $\tau = 0.05$, $\tau = 0.09$, and $\tau = 0.11$, and are shown in Fig. 5.13. For $\tau = 0.05$ and $\tau = 0.09$ the pore pressures are very similar to the ones measured on April 20th, 1965 (Fig. 5.11.) which is about 3 months after excavation. The time factor $\tau = 0.09$ represents a dissipation time of

$t = 0.6$ years for $c_s = 30$ ft^2/year, or

$t = 0.3$ years for $c_s = 60$ ft^2/year, the time factor $\tau = 0.05$ a time of

$t = 0.3$ years for $c_s = 30$ ft^2/year, or

$t = 0.15$ years for $c_s = 60$ ft^2/year.

For the intermediate state as well as for the state of full pore pressure equalization good agreement is indicated between field observations and analytical results. The variation of data depends mainly on the choice of the Coefficient of Swelling c_s and to a far lesser degree on the assumptions made for the actual analysis.

Slopes along the Panama Canal (USCE, 1970)

In a study of the slopes along the Panama Canal (USCE, 1970) previous records were reviewed, geological, field, and laboratory investigations were undertaken and the stability of the slopes was analysed. Piezometer installations indicated at several locations water pressure below canal level. These values show that negative excess pore pressures due to unloading during excavation still have not equalized. The measured pore pressures will be compared with the results from the analysis. The field data, however, are rather scarce. In most cases only one piezometer installation was reported for each cross-section. Further, the unloading history was most complicated for most slopes due to the successive number of landslides which had continuously changed the cross-section of the slopes. The "Model-Slope" at station 1796+00 is the only slope with piezometer installations that has not failed (See Fig. 5.14.). Because of its rather regular shaped profile this slope can be easily compared with the example given earlier. The following approximations have to be made:

1) The inclination of the analysed slope is 2 to 1, as compared with the actual average slope angle of 2.5 to 1.

2) The height H of the analysed embankment is 150 feet. This value is approximately equal to the difference between original groundsurface and present canal water level, as well as approximately equal to the height of the slope section below the level of the original groundwater table. The original groundwater table is believed to coincide with the lower boundary of the weathered zone.

The Coefficient of Swelling has been assumed to be $c_s \simeq 12$ ft^2/year, as compared to the reported laboratory value of $c_s = 1.2$ ft^2/year. The field value is believed to be substantially higher than the laboratory result, as indicated by the difference between field permeability and laboratory values of a factor of 10 to 100. Considering that the Coefficient of Volume Compressibility m_v is influenced far less, it is justified to assume that the c_s-value in the field is about 10 times larger than the reported laboratory value. The characteristic soil properties of the Cucaracha Shale at station 1796+00 are summarized in Table V.2.

The time for full pore pressure equalization has been found to be 700 years for $c_s = 12$ ft^2/year, or 250 years for $c_s = 25$ ft^2/year. The stabilized groundwater conditions have been assumed as indicated in Fig. 5.14. Thus, the presently observed excess pore pressure is about 50 feet below the equilibrium level. The analysis shows that this excess pore pressure is reached after a time factor of approximately 0.02. Thus, for a dissipation time of about 60 years the Coeffi-

cient of Swelling is c_s = 7.5 ft^2/year, which agrees with the previously estimated value. The increasing instability of the "Model-Slope", as indicated by the appearance of cracks, can be associated with the equalization of the negative excess pore pressures.

Coastal slope at Folkstone Warren, South England (Wood, 1971)

For the coastal slopes at Folkstone Warren in southern England pore pressures below sea-level were reported (Wood, 1971). The large landslides which have occurred at rather regular intervals at this location were described by Toms (1953), Wood (1955), and Hutchinson (1969). A cross-section of the 1915 slide is shown in Fig. 5.15. The sediments of Cretaceous age consist of chalk overlying Gault Clay, which rests on sands of the Folkstone beds. The back scarp is formed by the "High Cliff", beneath which the masses of an old slide extend as an irregular shelf.

During the past two centuries and perhaps over a longer period, there has been no general regression of the back scarp. Records of the slides exist only for renewed movements within the old slide masses. A comparison of the surface profiles before and after the large 1915 landslide indicates no substantial unloading of the embankment during the reactivated slide movements. It appears that the low water pressures in the slope have to be referred to the unloading during the initial slide.

The time for full equalization has been estimated by using the results of the present pore pressure analysis for an embankment

of 550 feet height, and a slope inclination of 2 to 1. The actual inclination of the slope at Folkstone Warren is on average 2.5 to 1. The time for full equalization of pore pressures has been computed to be t = 5000 years for c_s = 20 ft^2/year, or t = 2500 years for c_s = 40 ft^2/year. Assuming that the stabilized groundwater conditions are represented by the water level in the undisturbed Gault Clay at level +18 feet, the present negative excess pore pressures are equivalent to 50 feet water pressure. In the analysis this value is reached approximately after a time factor of τ = 0.05. Thus, the slide can be backdated by 750 years for c_s = 20 ft^2/year, or 375 years for c_s = 40 ft^2/year.

The occurrence of small subsidence movements in the chalk for a distance of around 60 feet behind the "High Cliff" has been quoted as evidence of progressive failure in the Gault Clay due to Bjerrum's mechanisms (Skempton and Hutchinson, 1969). The negative excess pore pressures in the slope suggest, however, that these movements can be associated with continuing pore pressure equalization.

Cuts in London Clay

Many cuts in London Clay failed about 20 to 60 years after they had been excavated. The failures were generally attributed to mechanisms of time-dependent strength reduction (e.g. Skempton, 1964; James, 1970). The results of the pore pressure analysis indicate for comparable slopes (H $\cong$ 80 feet, c_s $\cong$ 40 ft^2/year) a time for full equalization of 60 years. This result may suggest that the slides

in London Clay could also be explained in terms of delayed pore pressure equalization.

Welland Test Pit, Canada (Kwan, 1971)

As a part of the stability study for the design of side slopes of a canal a comprehensively instrumented test excavation was made in Welland clays, which are overconsolidated clays of Pleistocene age. The cut was made in a clayey silty till, which was underlain by stratified clay, as shown in Fig. 5.16. The material properties of the clays of the test slope are summarized in Table V.3.

Excavation was carried out in stages. In the first stage the top 17 feet of dessicated sediments were removed bringing the ground level to elevation 560.0 feet. In the second stage a vertical slope was cut within 4 days to an average depth of 30 feet. The length of the slope was limited to 50 feet by excavating two vertical trenches at the ends, as indicated in Fig. 5.17. During the entire excavation period precipitation was low and temperatures rarely dropped below 0^{o} F. On February 21st, a sump was excavated on top of the vertical block and water was ponded in the sump for a short time in an attempt to break the frozen surface. After that a hair line crack was discovered which quickly developed across the sump at a distance of approximately 21 feet from the vertical face (See Fig. 5.17.). On February 22nd failure occurred, after the slope had stood vertically for 4 days. The failure plane did not follow the initially observed tension crack, but started along a second crack which became visible only hours before the failure took

place, as shown in Fig. 5.17.

After the completion of the first stage of excavation the anticipated failure area was instrumented. Marker stakes were driven, strain recording devices were installed, and 52 piezometers open standpipes of Geonor-type were installed in two identical rows parallel to the center line of the excavation. The location of the piezometers around the failure area is shown in Fig. 5.16. The pore pressure readings immediately before excavation of the vertical slope (Febr.15th) are given in Fig. 5.18., and the readings taken 5 days after the end of construction of the vertical slope (Febr. 23rd, 1967) in Fig. 5.24.

For the calculation of the pore pressure changes due to excavation a Finite Element model is used as shown in Fig. 5.19. For the region of interest a finer grid is selected than for the surrounding zones. To simulate the unloading process stresses are applied as shown in Fig. 5.20. The Coefficient of Earth Pressure at Rest K_o is unknown, and has been assumed to be 1.0 and 1.4. For the Welland Canal an A-value of 0.7 had been reported for loading in a triaxial test. The vertical cut under consideration was separated from the adjoining zones by vertical trenches. Thus, unloading in the field occurred under conditions similar to a triaxial test, and the reported A-value might be a realistic value for this slope configuration. The pore pressure changes due to unloading are analysed for A = 0.7 and A = 0.3. In Fig. 5.16. the pore pressure changes from the analysis are compared with the measured values. The measured decrease of pore pressures from the value before excavation

to a constant value, around 4 to 5 days after the end of construction, is considered to represent the pore pressure changes due to excavation. The calculated pore pressures for the stage immediately after excavation are shown in Figs. 5.21., 5.22., and 5.23. The measured pore pressures after excavation are given in Fig. 5.24.

Failure of the vertical face occurred 2 days after completion of the excavation. Because very little pore pressure equalization had occurred within this short time period, it is not possible to correlate numerically the failure with the dissipation of negative excess pore pressures. For the determination of the excess pore pressures steady seepage conditions have been assumed as indicated in Fig. 5.25. Complete dissipation of the excess pore pressures has been obtained for a time factor $\tau = 0.03$. With a reported Swelling Coefficient c_s = 30 ft^2/year the time for full equalization of pore pressures is about one year.

The average strength mobilized at failure of the cut was reported as close to peak. The failed material shows a very flat-topped stress strain curve, which indicates that the average strength mobilized at first-time failure in this material has to be close to peak strength. If the calculated pore pressures are in good agreement with the actual values, a stability analysis, which considers the computed pore pressures and peak strength parameters should result in a Factor of Safety equal to one.

Two failure surfaces are analysed,

a) the failure surface which considers the maximum extension of

the failure zone, as indicated by section a) in Figs. 5.17. and 5.26.,

b) the failure surface which considers the minimum extension of the failure zone, as indicated by section b) in Figs. 5.17. and 5.26.

For the computed pore pressure distribution as shown in Fig. 5.21. the Factor of Safety is 0.86 along section a) and 1.10 along section b). The average Factor of Safety is 1.00. The stability analysis shows that the computed pore pressures in the failure zone are in good agreement with the actual values.

Pore pressures immediately after excavation were reported for an excavation in Boom Clay (Morgenstern, 1971) with dimensions similar to the cut in Welland Clay. The measured pore pressure distribution immediately after excavation, as shown in Fig. 5.27., indicates a pattern comparable to the computed pore pressures, as shown in Figs. 5.21. and 5.22.

4. Discussion

The analysis of the equalization of pore pressures after excavation of a cut is affected by several limitations and assumptions:

1) A purely elastic material has to be assumed.
2) Generally the pore pressure parameter A for unloading is not well known.
3) A Coefficient of Earth Pressure at Rest K_o has to be assumed.

4) Pore water distribution before excavation and the final steady flow conditions have to be assumed for most cases.

5) The overall Coefficient of Swelling c_s for a natural material is generally not clearly defined.

6) The two-dimensional consolidation analysis implies serious limitations, particularly because drainage boundary conditions along the base of a cut are not clearly known in most natural slopes. The change of permeability due to the change of effective stresses cannot be considered in the analysis.

The predictions of pore pressures immediately after excavation, however, appeared in rather good agreement with field observations, particularly for the slope of the Kimola Canal, but also for the Welland Test Pit and the cut in Boom Clay. It seems that the pattern of pore pressures within the slope region is relatively insensitive to variations of K_o- and A- values, at least for the range of values investigated

The pore pressure analysis based on the Finite Element Analysis may become a valuable tool to determine the pore pressures in the ground immediately after excavation of a slope in homogeneous overconsolidated materials. However, for this application of the analysis further comparison with good field data is needed.

The modelled process of pore pressure dissipation can be compared with field conditions only in very general terms, mainly because of the lack of continuous pore pressure observations after excavation

of a cut. The few data obtained, however, indicate its practical value: The observations from the Kimola Canal are in surprisingly good agreement with the analytical results. Also the analysis of the "Model-Slope" along the Panama Canal shows reasonable agreement with the field observations. The time range for full dissipation of excess pore pressures due to unloading of higher slopes is in the order of many hundred years. Therefore many failures might be attributed to delayed pore pressure equalization.

It is interesting to note that the computed time values for pore pressure equalization are of the same order of magnitude as the values which had been reported for the time between excavation and failure of many comparable slopes. This observation might suggest that many slides which in the past have been explained by progressive failure could have been caused by the decrease of effective stress due to dissipation of negative excess pore pressures.

TABLE V.1.

KIMOLA CANAL: SUMMARY OF SOIL PROPERTIES AT CANAL SECTION ST. 52+70 (AFTER KANKARE, 1968)

Clay Fraction $\%<2\mu$	w_N	w_p	w_L	I_p	I_L	Cu	Precon-solida-tion Load	Sensitiv-ity	c' peak	ϕ' peak	Cc
%	%	%	%	%		t/m^2	Kg/cm^2		Kg/cm^2	degrees	$Ft^2/year$
61.0	53.0	26.0	55.6	30.0	.87	3	.05	.16	.049	27.7	30

TABLE V.2

PANAMA CANAL: SUMMARY OF SOIL PROPERTIES OF CUCARACHA SHALE

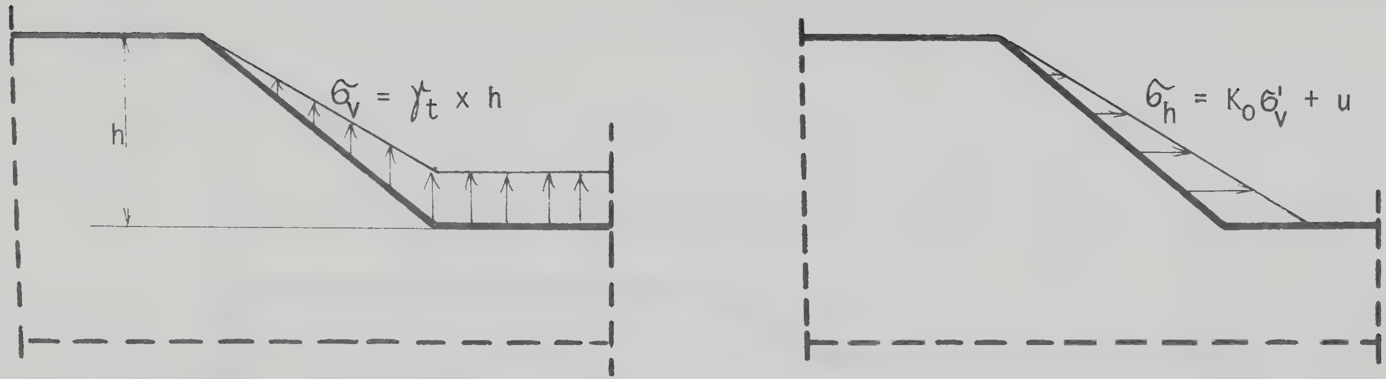
OF MODEL SLOPE AT STATION 1796 + 00 (AFTER USCE, 1970)

w_N	w_p	w_L	I_p	A	ϕ' peak	c' peak	ϕ' residual	Results from Lab. Tests K	Results from Lab. Tests Cv	Result from Field Test K
%	%	%	%		degrees	t/sft.	degrees	cm/sec.	ft^2/year	cm/sec.
15	29	65	36	1.2	18	0.2	8	10^{-10}	1.2	10^{-8} to 10^{-9}

TABLE V.3.

WELLAND TEST PIT: SUMMARY OF SOIL PROPERTIES (AFTER KWAN, 1971)

	w_N	Clay Fraction	K	Cv	Plasticity	c'_{peak}	ϕ'_{peak}
		$\% < 2\mu$	cm/sec.	ft^2/day		psf	degree
Till	26	45	$<10^{-7}$	.08	medium	280	24.5
Stratified Clay	40	50	$<10^{-7}$	.12	medium to high	280	22.6



Vertical Unloading Stresses

Horizontal Unloading Stresses

FIG. 5.1. PORE PRESSURES DUE TO UNLOADING: UNLOADING STRESSES UPON EXCAVATION.

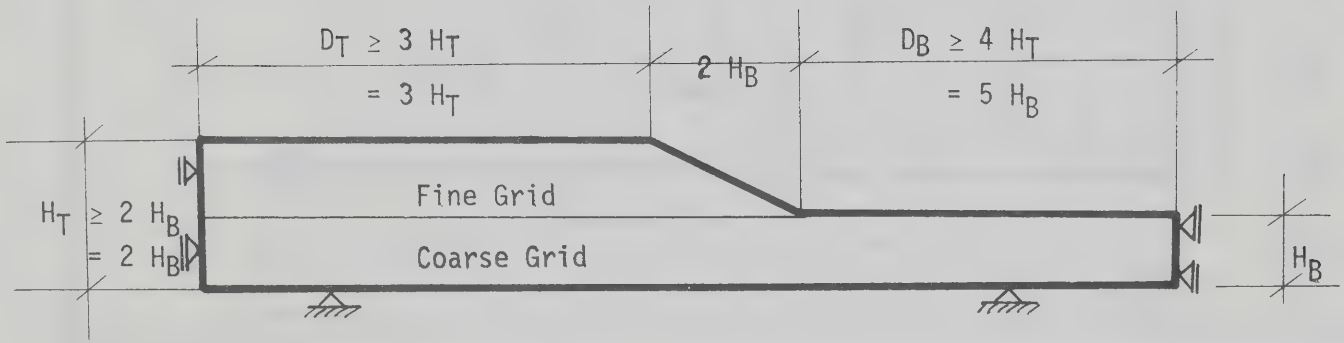


FIG. 5.2. PORE PRESSURES DUE TO UNLOADING:
SYSTEM USED FOR THE STRESS ANALYSIS WITH FINITE ELEMENTS.

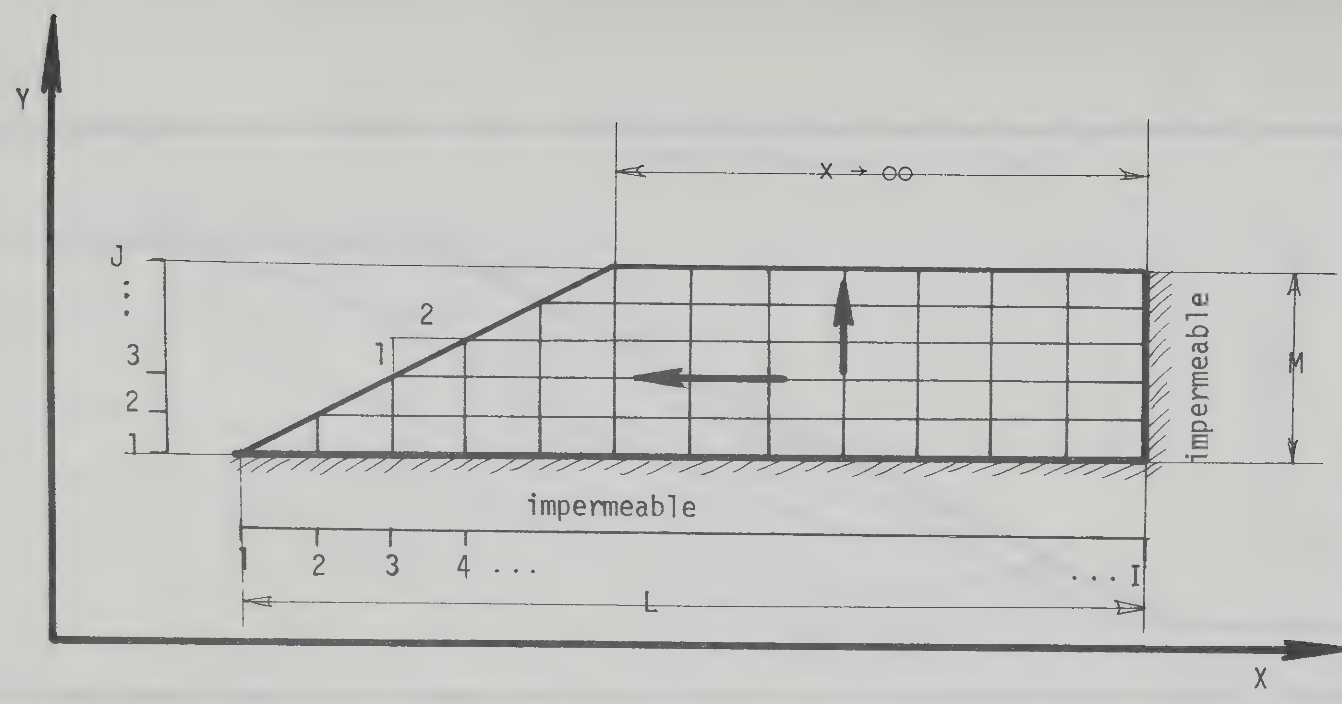


FIG. 5.3. PORE PRESSURE EQUALIZATION:
SYSTEM FOR THE TWO-DIMENSIONAL CONSOLIDATION ANALYSIS.

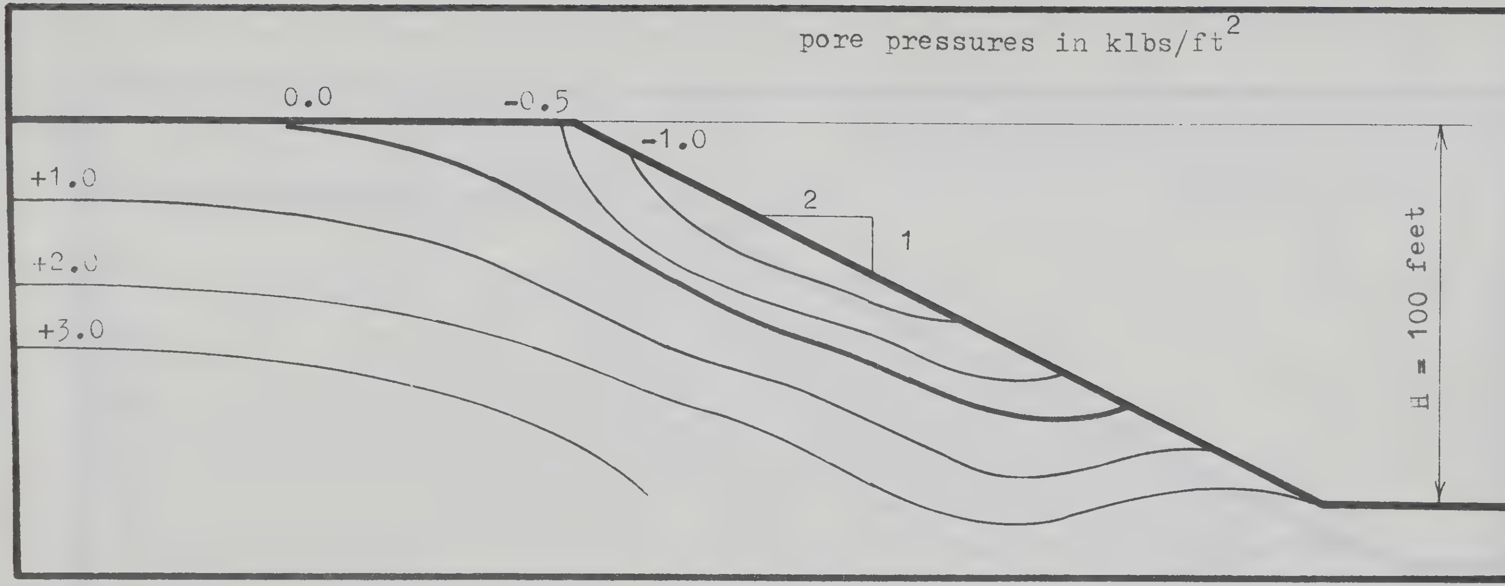


FIG. 5.4. PORE PRESSURES DUE TO UNLOADING:
PORE WATER DISTRIBUTION IMMEDIATELY AFTER EXCAVATION
OF A 2:1 SLOPE, 100 FEET DEEP, FOR A SOIL WITH $K_0 = 1.0$, $A = 1/3$.

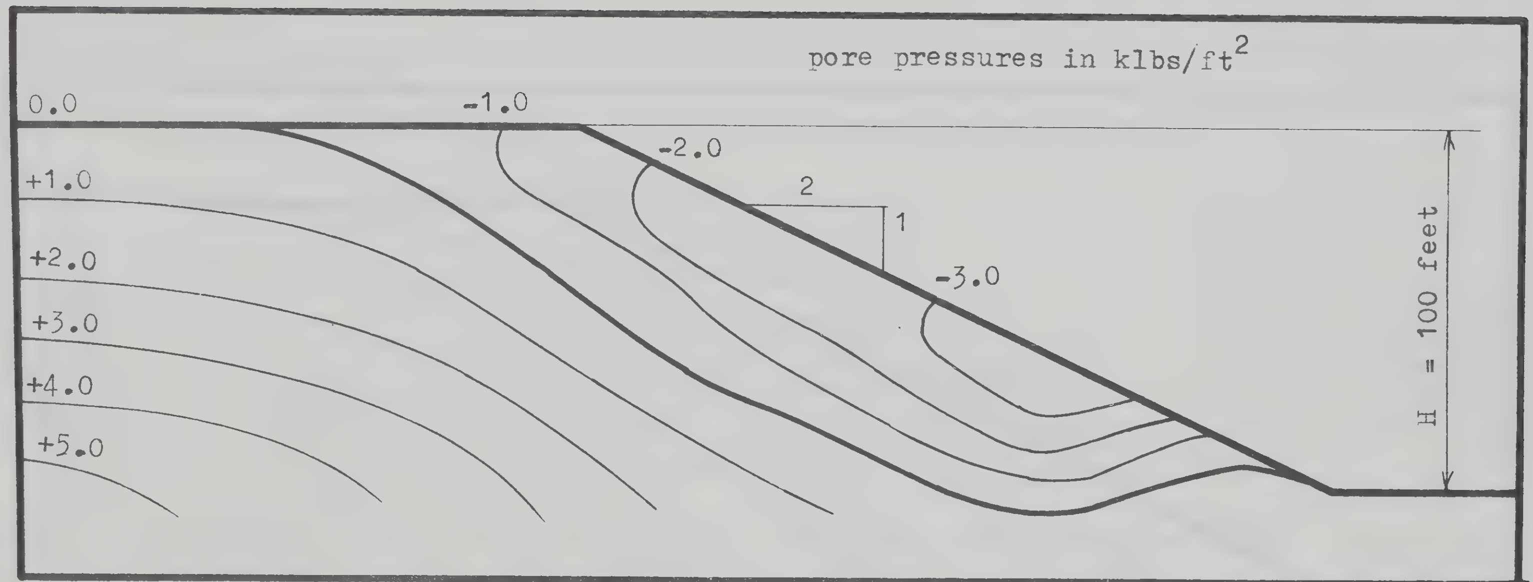


FIG. 5.5. PORE PRESSURES DUE TO UNLOADING:
PORE WATER DISTRIBUTION IMMEDIATELY AFTER EXCAVATION
OF A 2:1 SLOPE, 100 FEET DEEP, FOR A SOIL WITH K_0 = 1.5, A = 1/3.

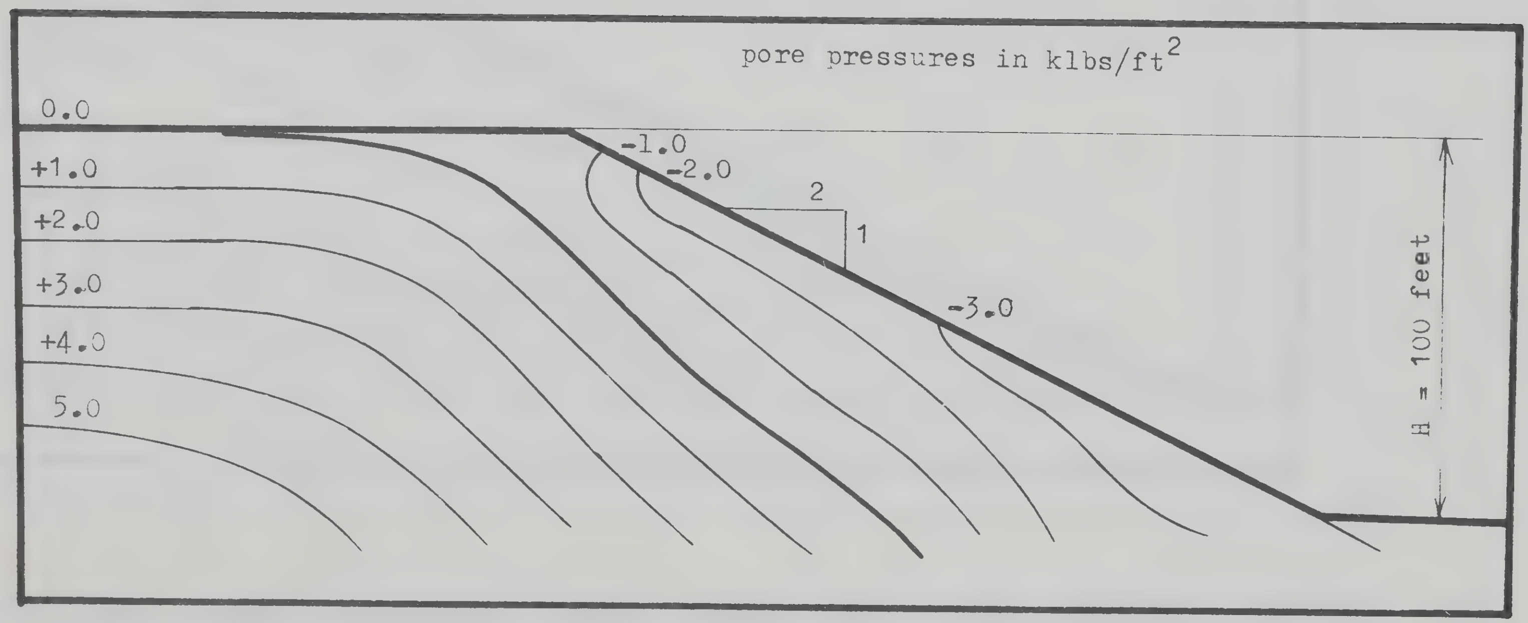


FIG. 5.6. PORE PRESSURES DUE TO UNLOADING:
PORE WATER DISTRIBUTION IMMEDIATELY AFTER EXCAVATION
OF A 2:1 SLOPE, 100 FEET DEEP, FOR A SOIL WITH K_o = 1.5, A = 0.

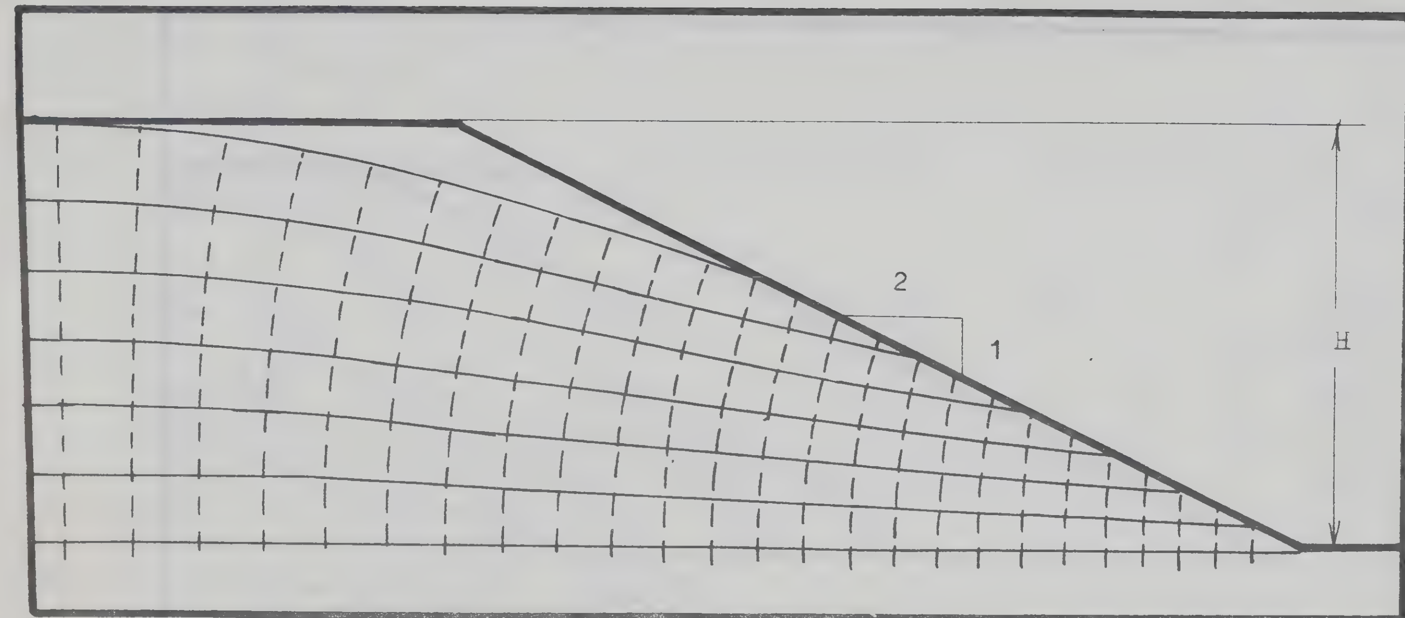


FIG. 5.7. PORE PRESSURE EQUALIZATION: FLOW NET REPRESENTING STABILIZED GROUNDWATER CONDITIONS.

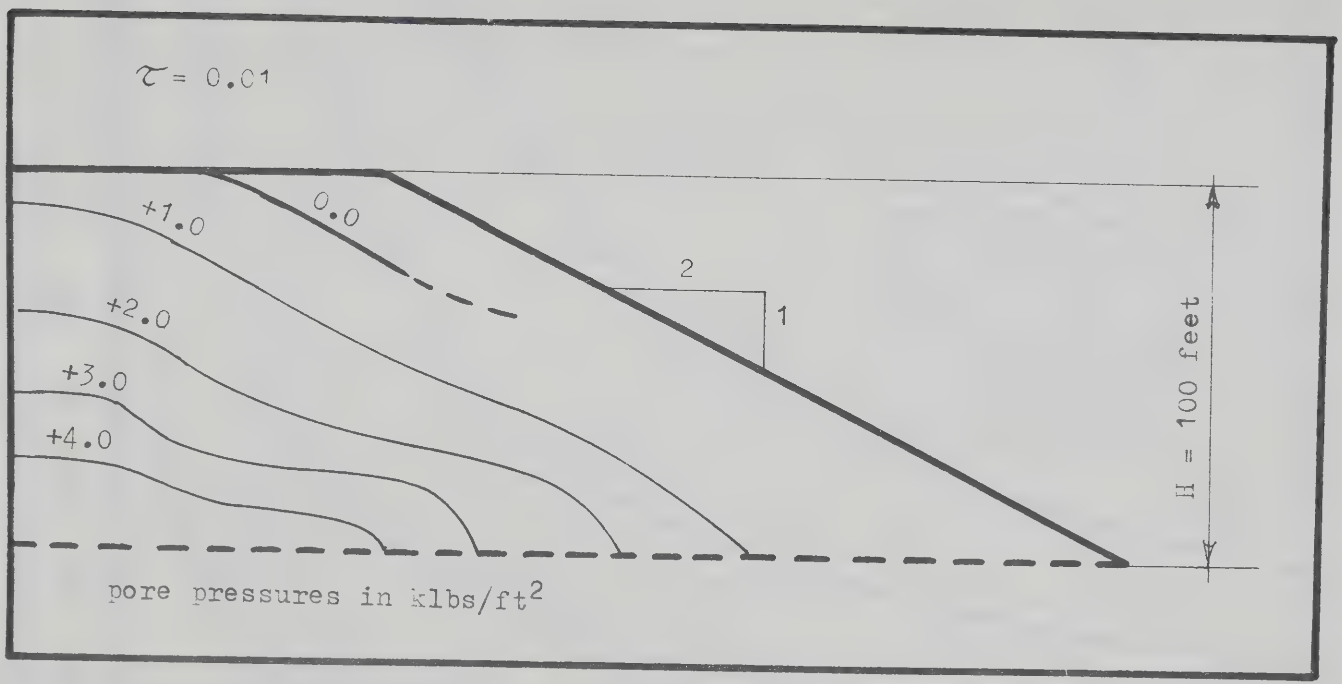


FIG. 5.8. DISSIPATION OF EXCESS PORE PRESSURES:
PORE PRESSURE DISTRIBUTION AFTER TIME FACTOR τ = 0.01.

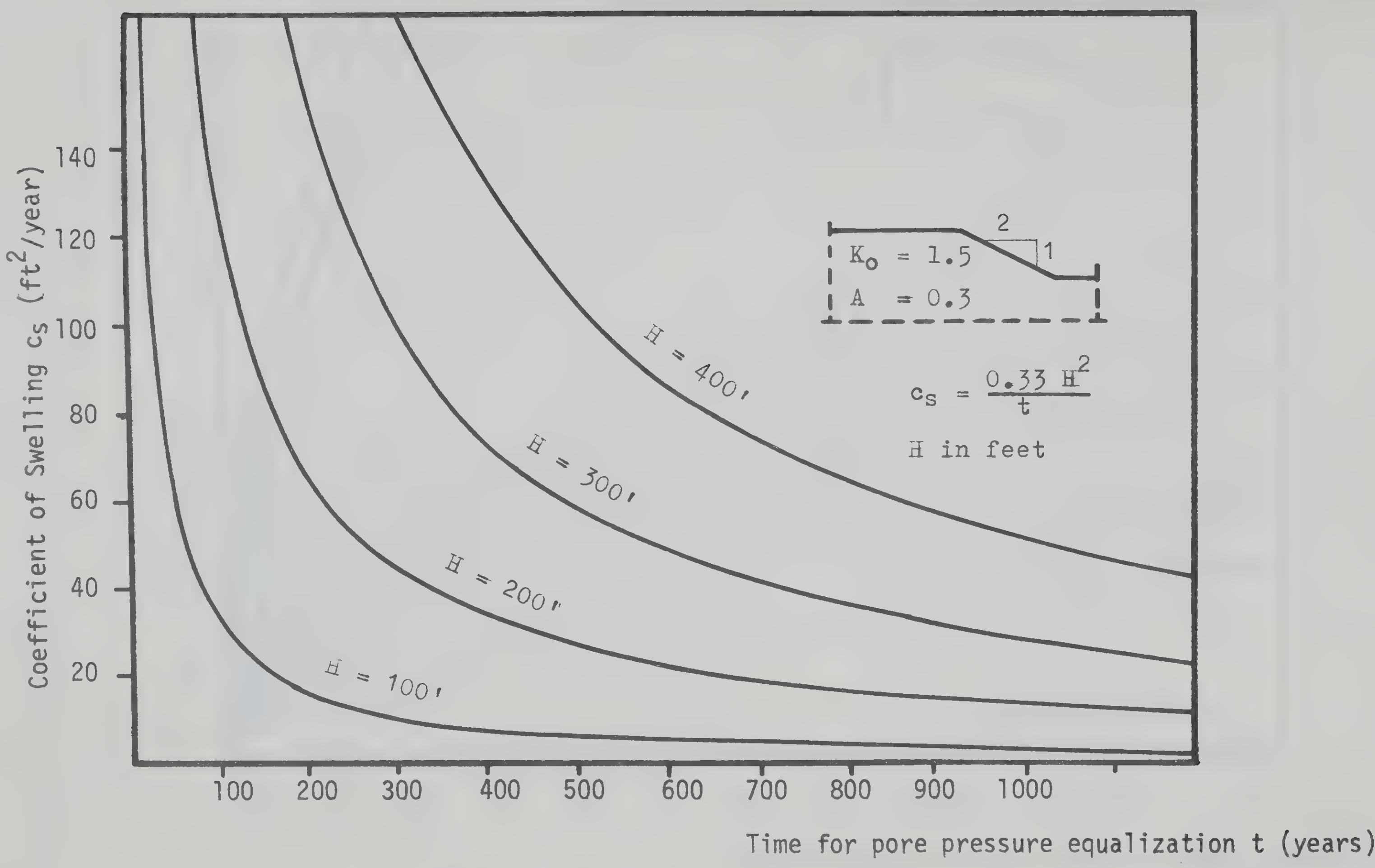


FIG. 5.9. PORE PRESSURE EQUALIZATION:
COEFFICIENT OF SWELLING VERSUS TIME FOR PORE PRESSURE EQUALIZATION
FOR CONSTANT SLOPE HEIGHT.

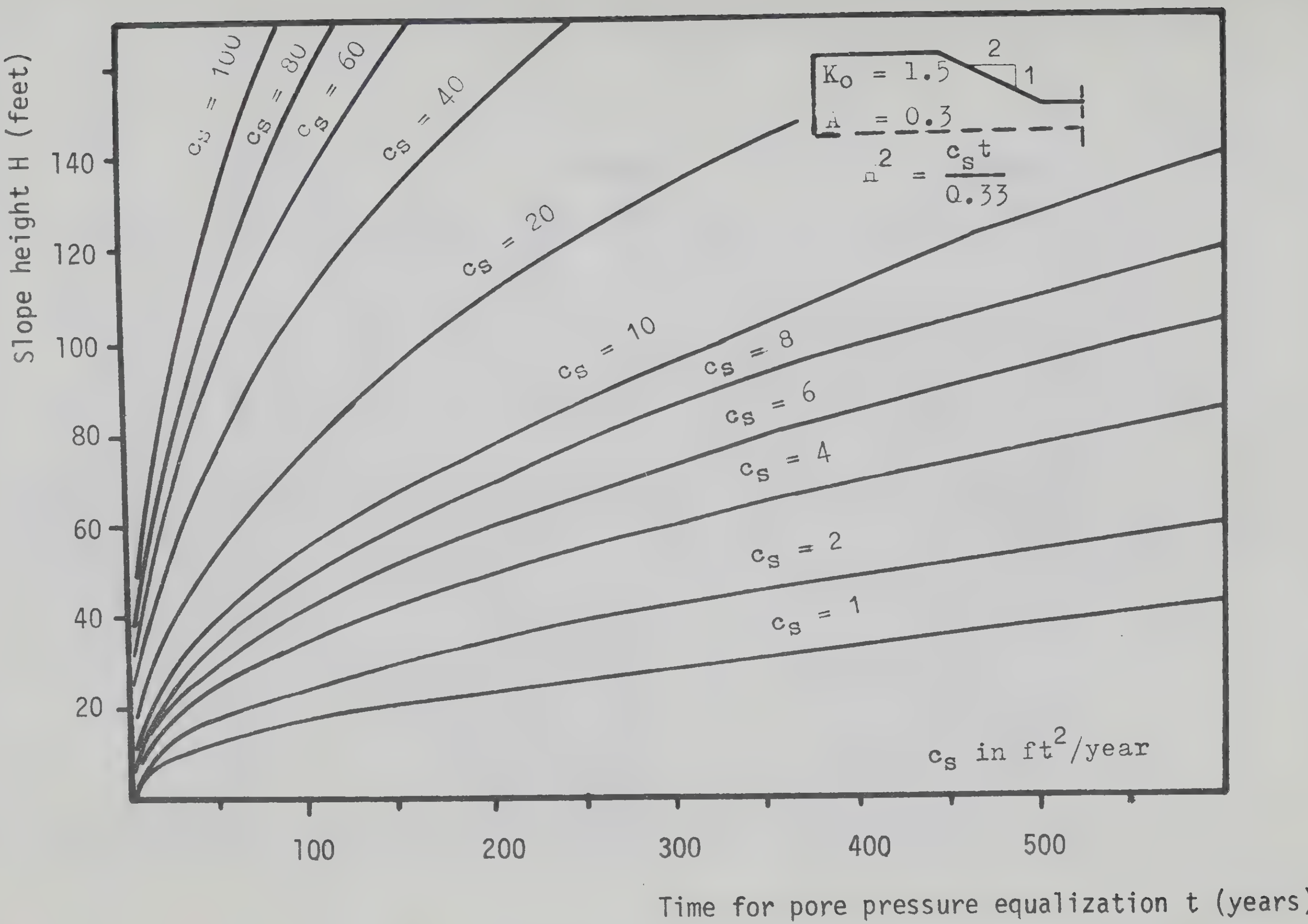


FIG. 5.10. PORE PRESSURE EQUALIZATION:
SLOPE HEIGHT VERSUS TIME FOR PORE PRESSURE EQUALIZATION
FOR CONSTANT COEFFICIENT OF SWELLING.

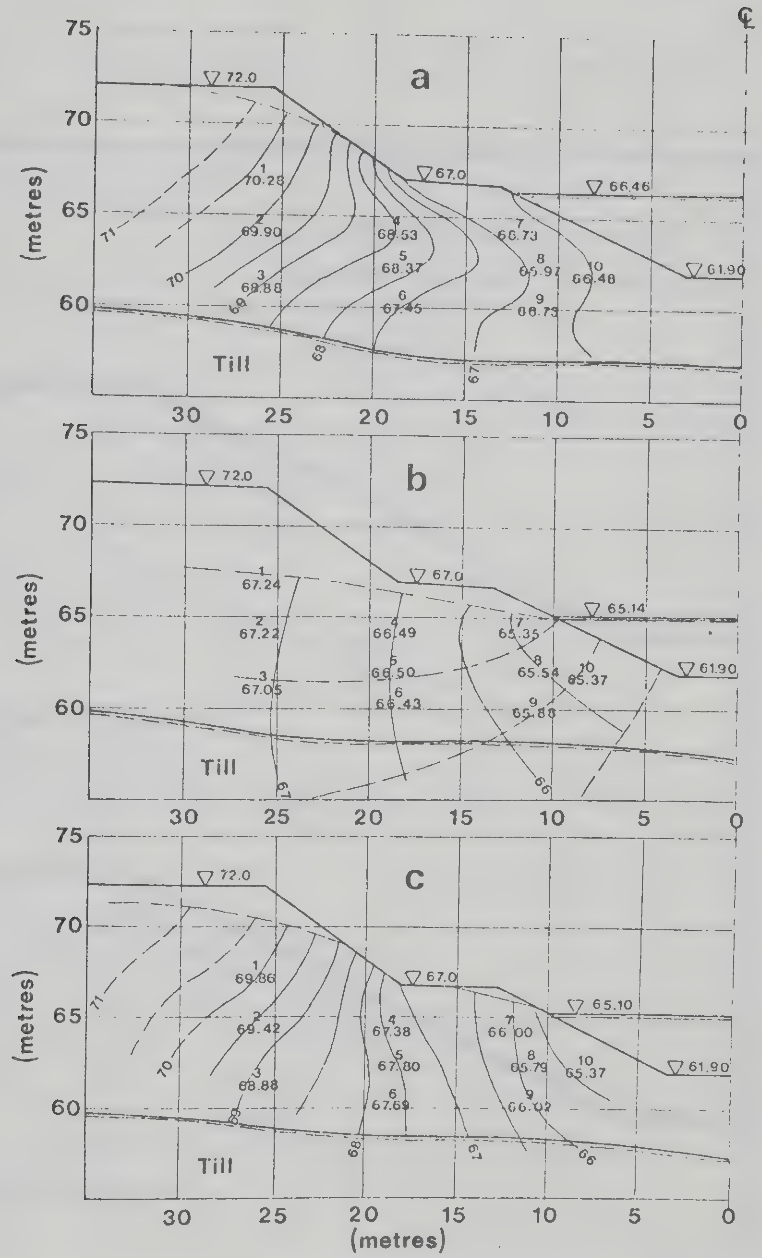


FIG. 5.11. KIMOLA CANAL:
EQUIPOTENTIAL CURVES OF PORE PRESSURE AT STATION 52+70.
a) APRIL 20th, 1965
b) JULY 25th, 1965
c) OCTOBER 27th, 1965

(KANKARE, 1968)

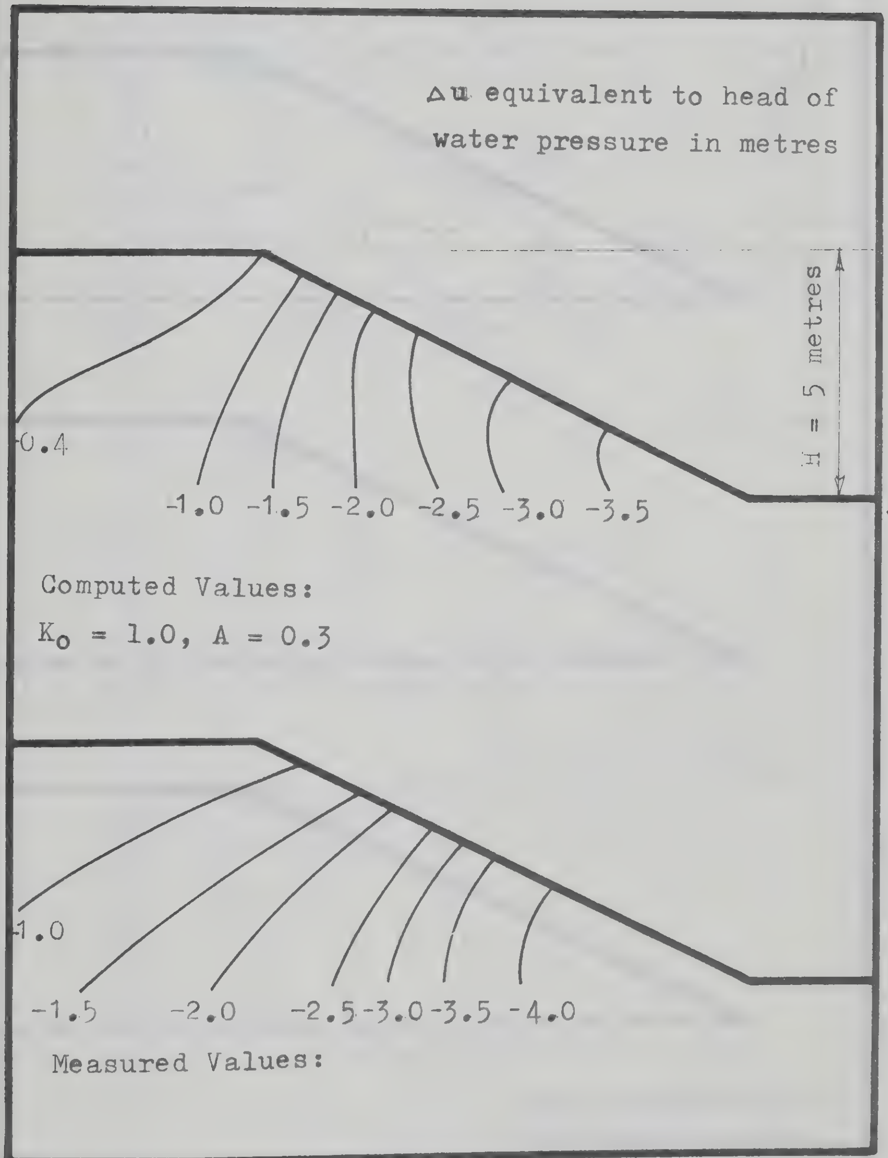


FIG. 5.12.
CHANGE OF PORE PRESSURES DUE TO EXCAVATION OF KIMOLA CANAL SECTION 52+70:
COMPARISON OF COMPUTED AND MEASURED PORE PRESSURE CHANGES
(AFTER KANKARE, 1968).

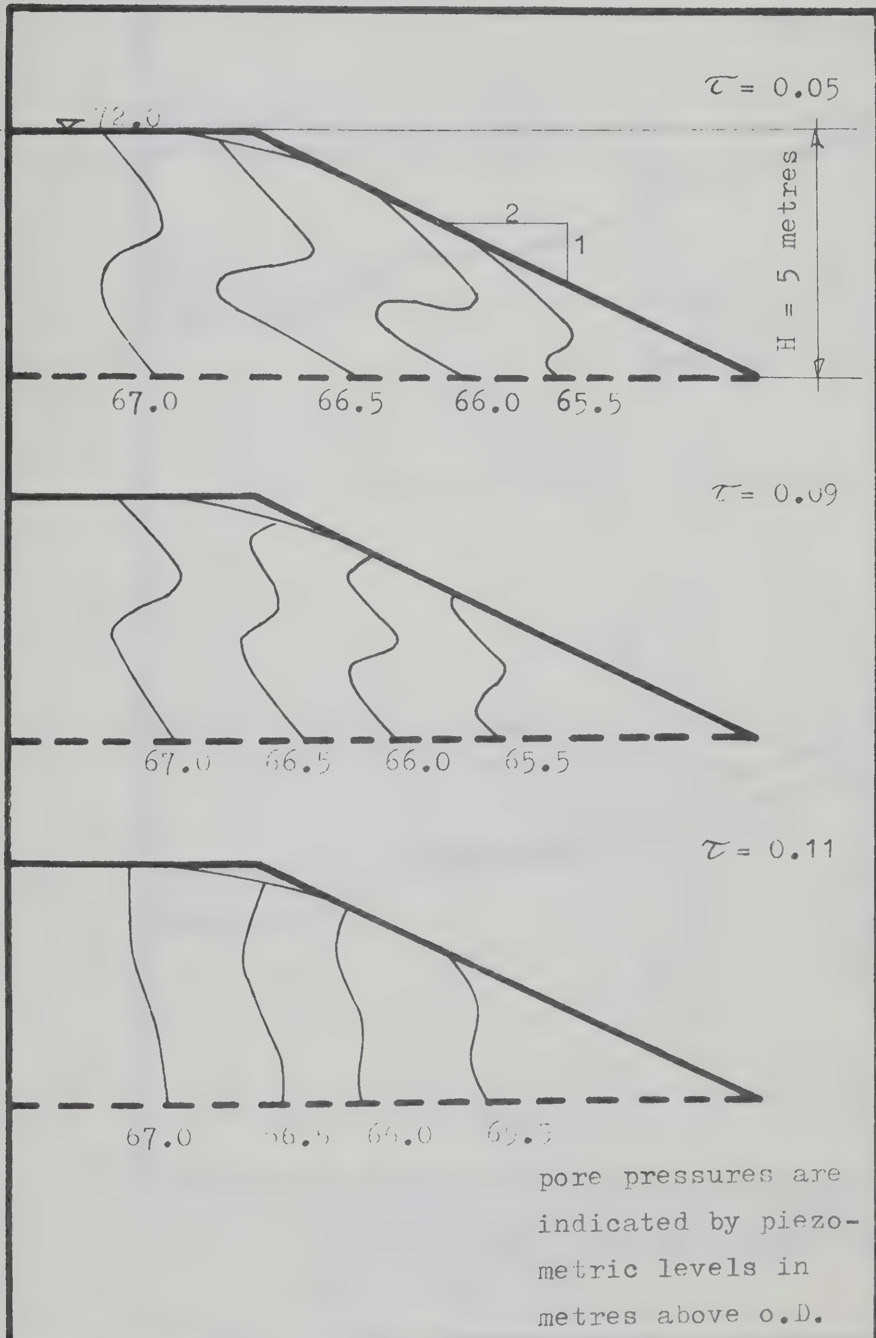


FIG. 5.13.
DISSIPATION OF EXCESS PORE PRESSURES IN KIMOLA CANAL SLOPE AT ST. 52+70: PORE PRESSURE DISTRIBUTION AT 3 INTERMEDIATE STAGES OF DISSIPATION.

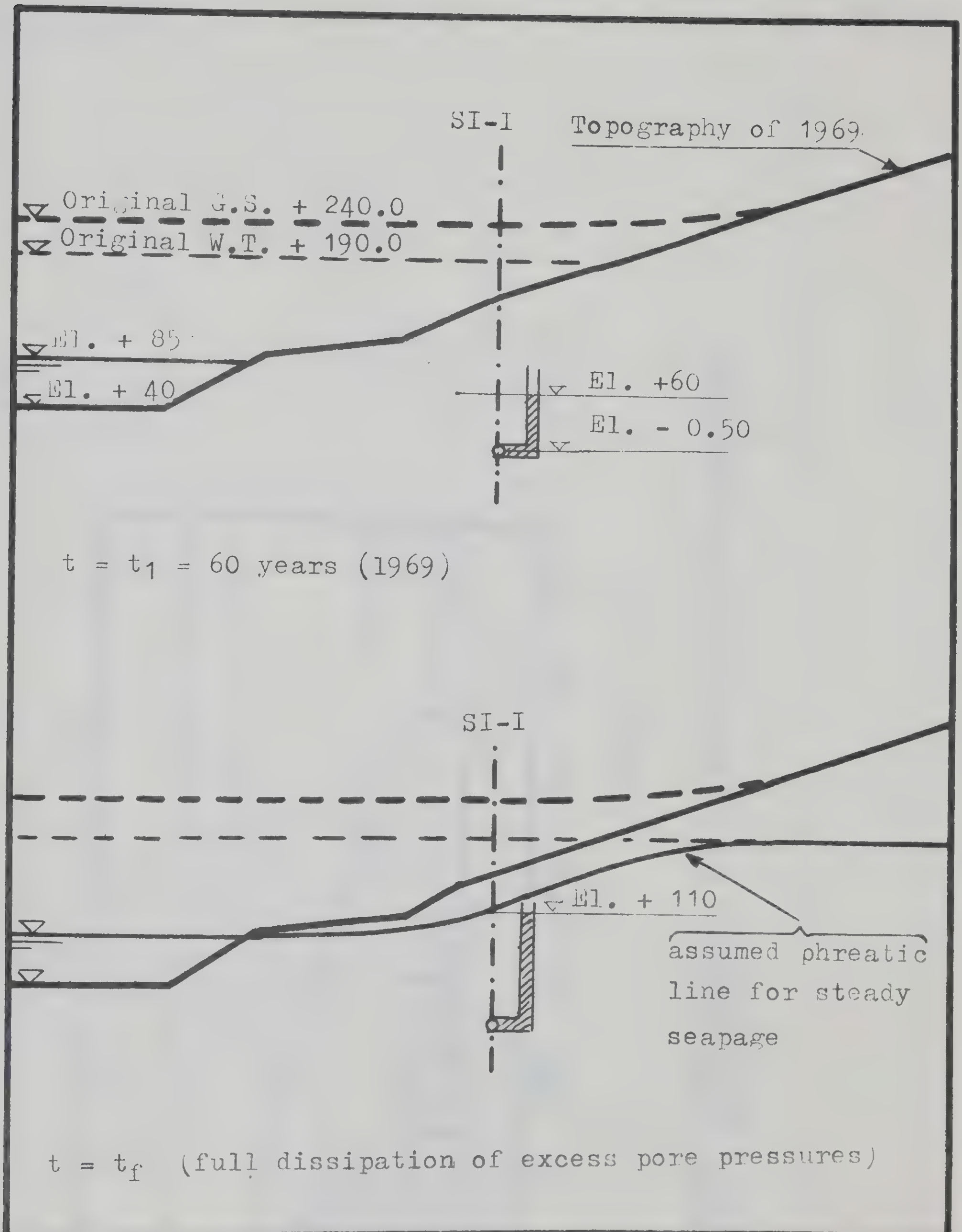


FIG. 5.14. DISSIPATION OF EXCESS PORE PRESSURES AT PANAMA CANAL, STATION 1796+00 ("MODEL SLOPE") (AFTER USCE, 1970).

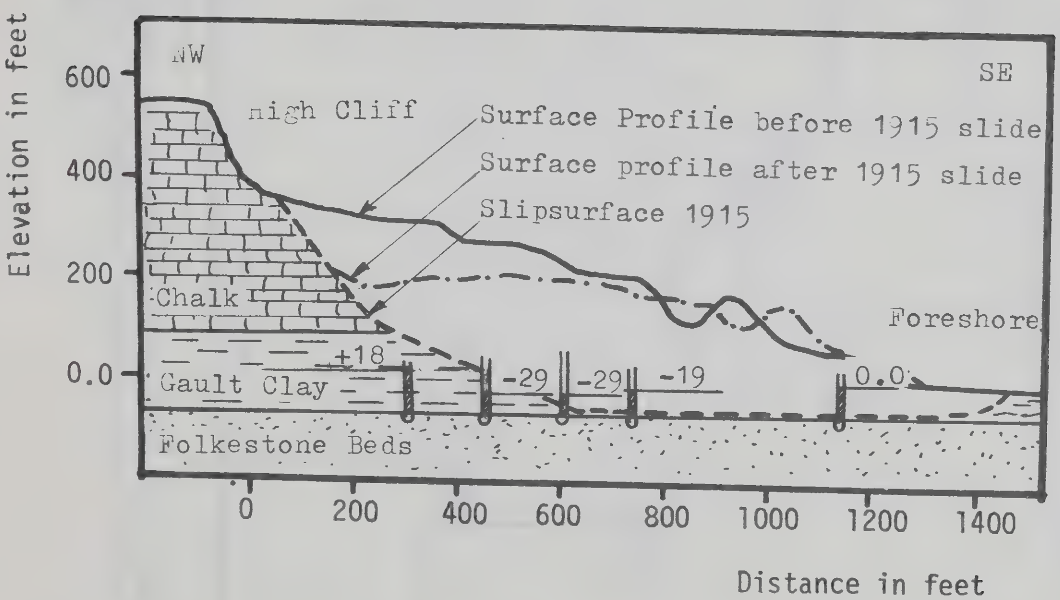


FIG. 5.15. NEGATIVE PORE PRESSURES BELOW LANDSLIDE AT FOLKESTONE WARREN (AFTER WOOD, 1971).

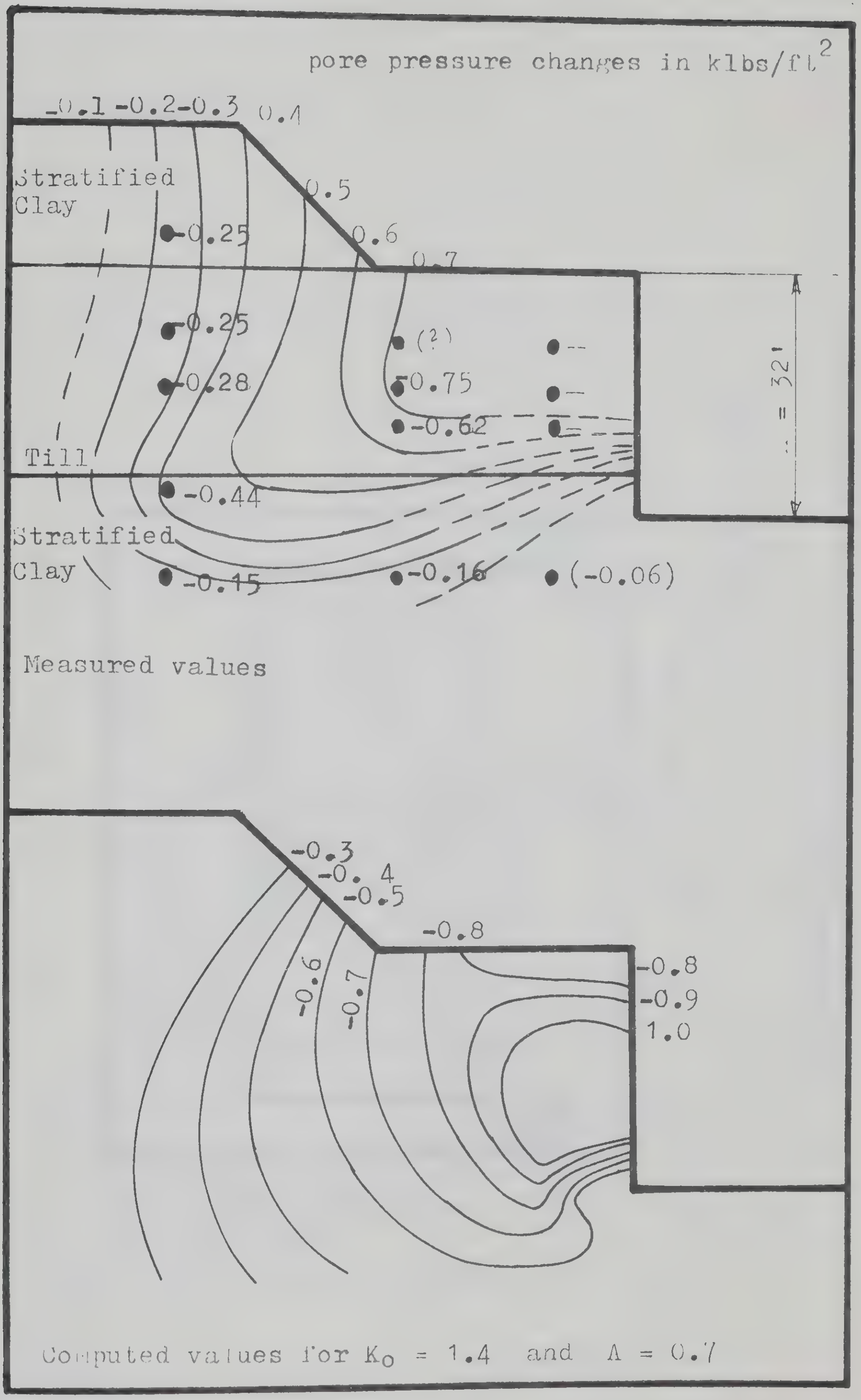


FIG. 5.16.
CHANGE OF PORE PRESSURES DUE TO EXCAVATION OF WELLAND TEST PIT: COMPARISON OF MEASURED AND COMPUTED PORE PRESSURE CHANGES (AFTER KWAN, 1971).

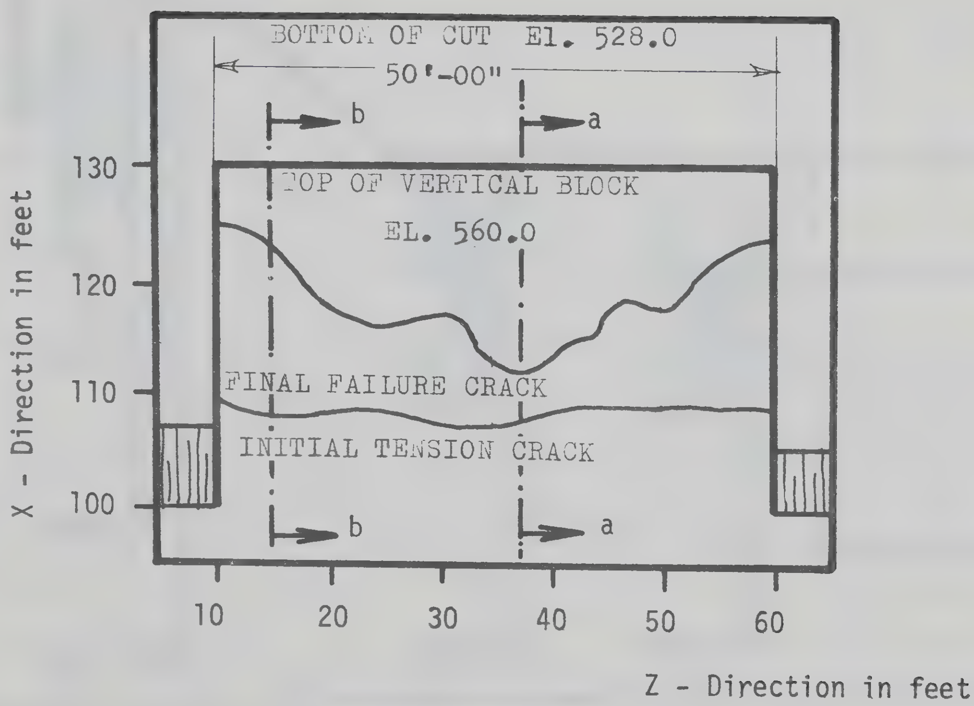


FIG. 5.17. WELLAND TEST PIT:
LOCATION OF TENSION CRACKS (AFTER KWAN, 1971).

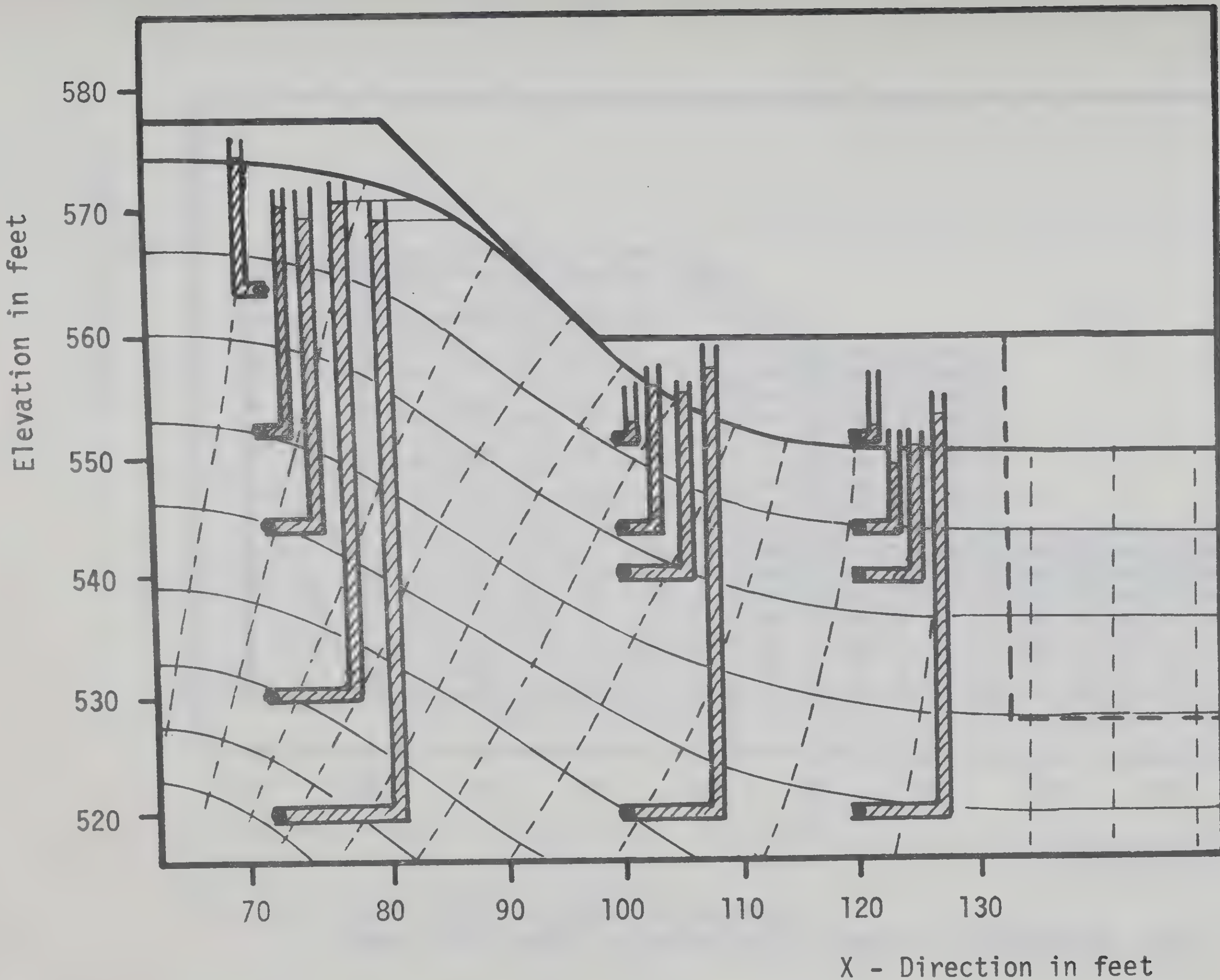


FIG. 5.18. WELLAND TEST PIT:
FLOW NET CONSTRUCTED ACCORDING TO THE PRESSURE READINGS IMMEDIATELY BEFORE CONSTRUCTION OF TRIAL PIT (KWAN, 1971).

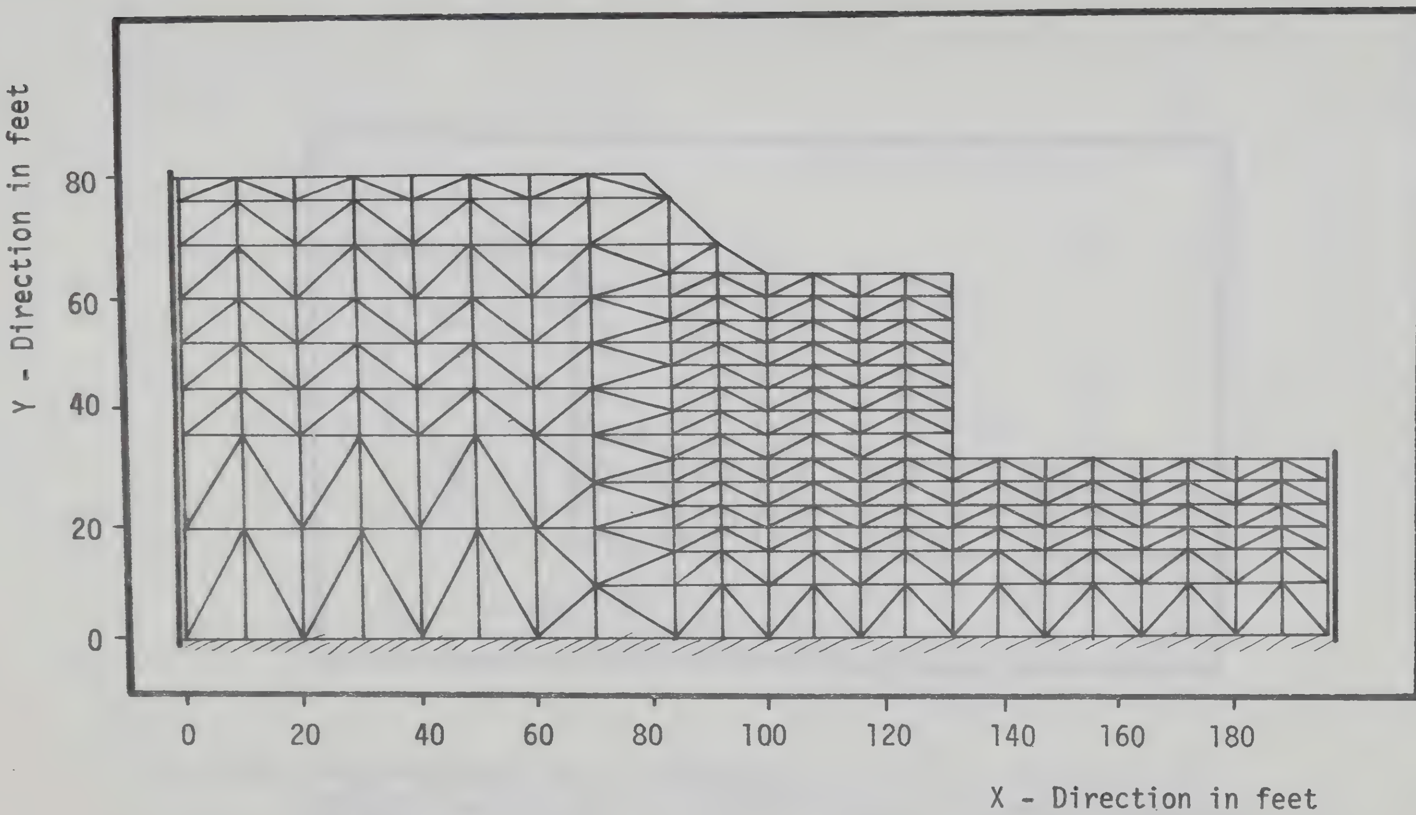


FIG. 5.19. PORE PRESSURES DUE TO UNLOADING:
MODEL FOR FINITE ELEMENT STRESS ANALYSIS OF WELLAND TEST PIT.

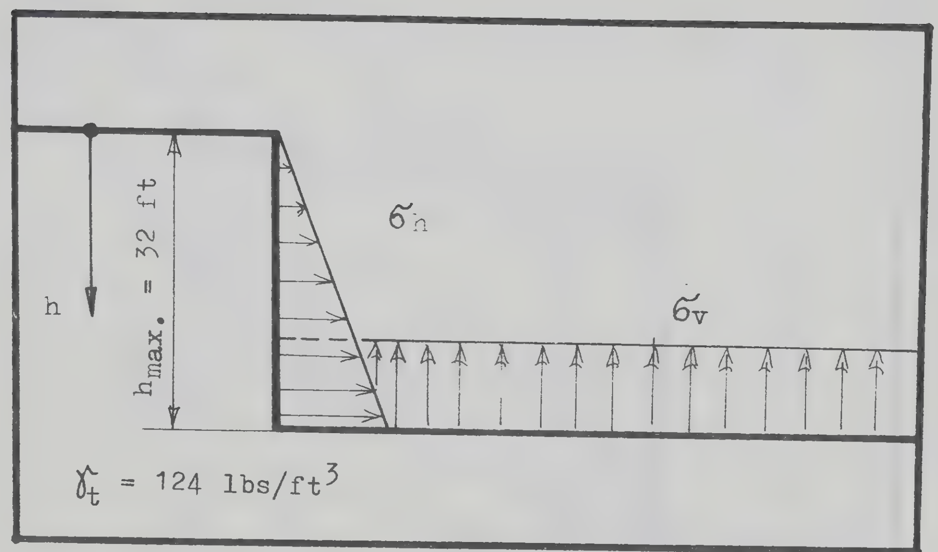


FIG. 5.20. PORE PRESSURES DUE TO UNLOADING:
STRESSES APPLIED TO THE EXCAVATED FACES TO SIMULATE UNLOADING FOR WELLAND TEST PIT.

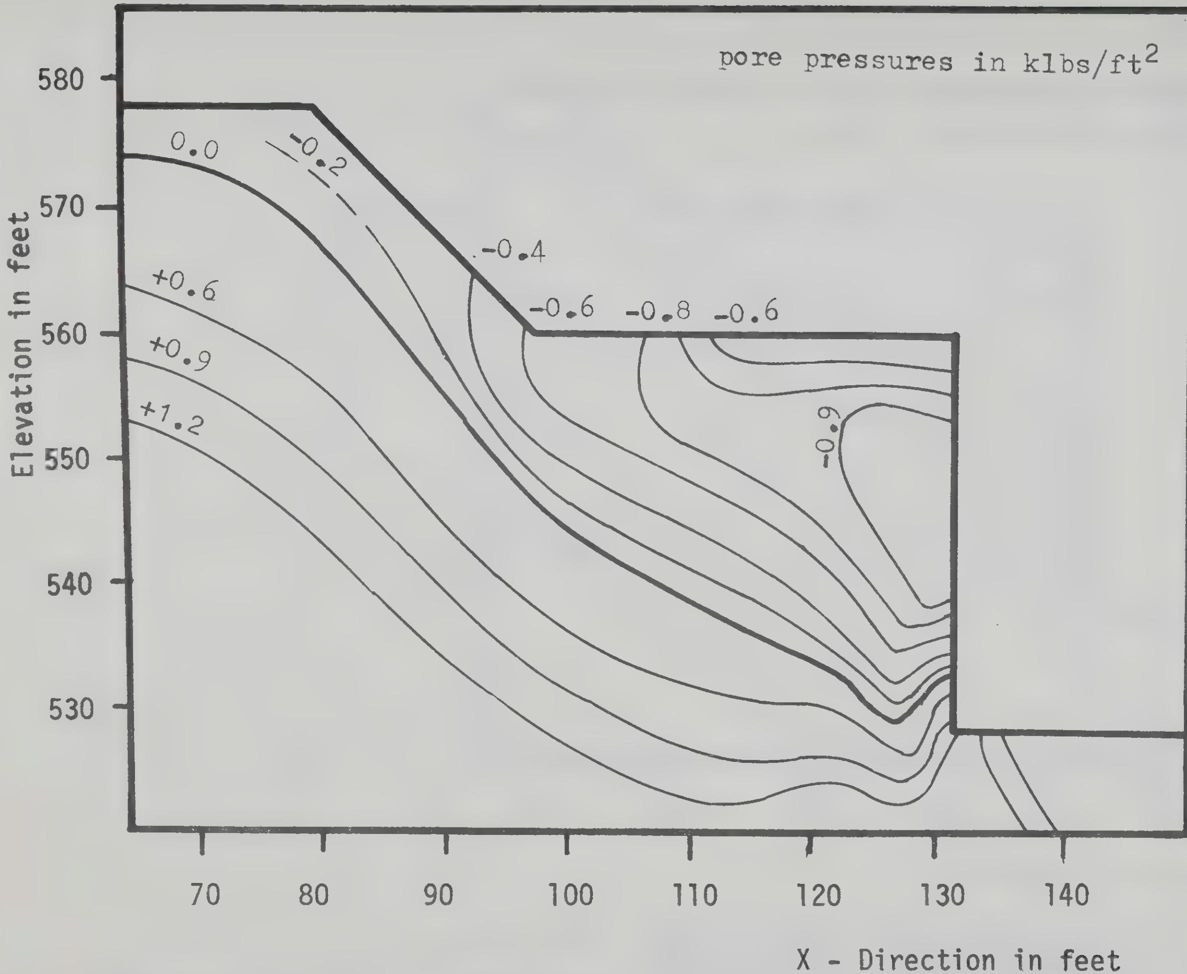


FIG. 5.21. PORE PRESSURES DUE TO UNLOADING:
COMPUTED PORE PRESSURES IMMEDIATELY AFTER EXCAVATION OF WELLAND TEST PIT FOR $K_o = 1.4$, $A = 0.7$.

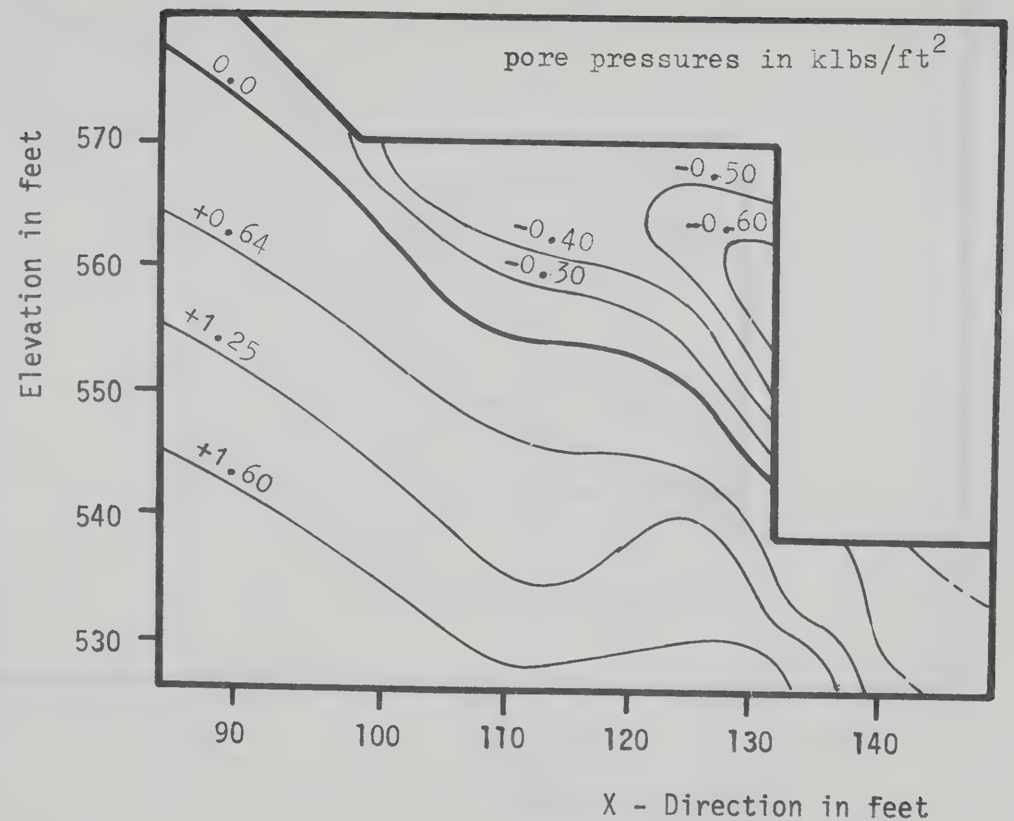


FIG. 5.22. PORE PRESSURES DUE TO UNLOADING: COMPUTED PORE PRESSURES IMMEDIATELY AFTER EXCAVATION OF WELLAND TEST PIT FOR $K_o = 1.4$, $A = 0.3$.

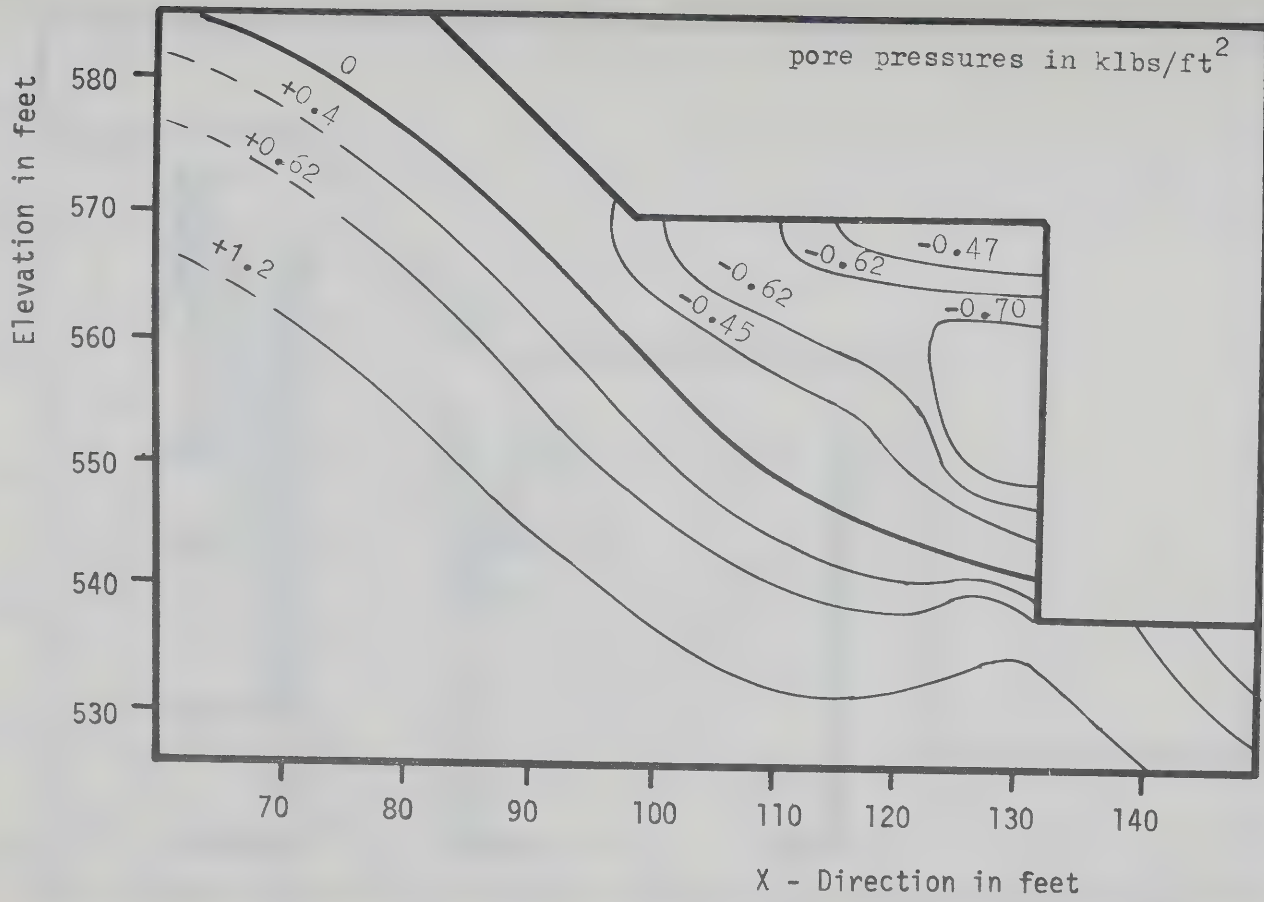
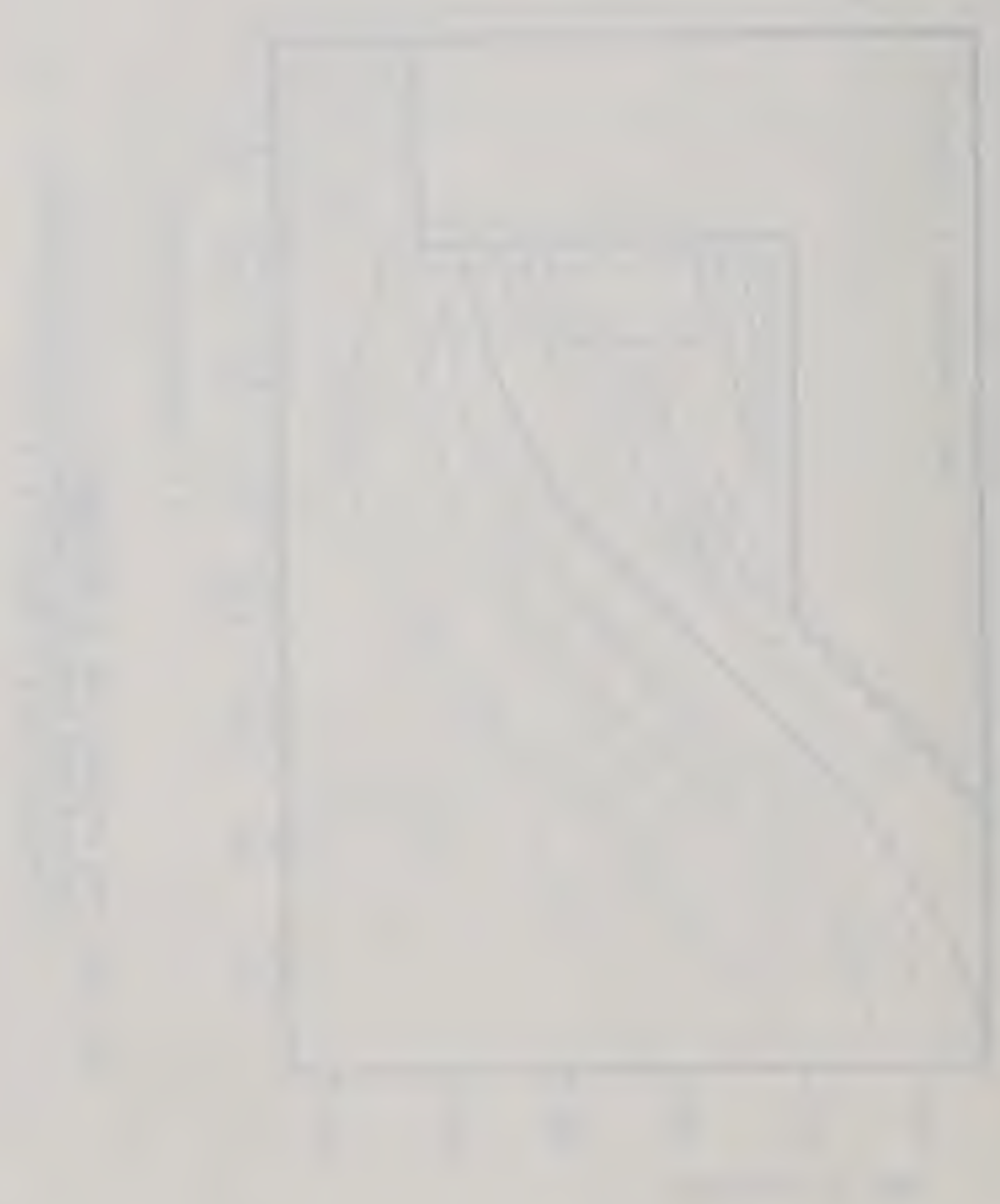


FIG. 5.23. PORE PRESSURES DUE TO UNLOADING:
COMPUTED PORE PRESSURES IMMEDIATELY AFTER EXCAVATION OF WELLAND TEST PIT FOR $K_o = 1$, $A = 0.7$.



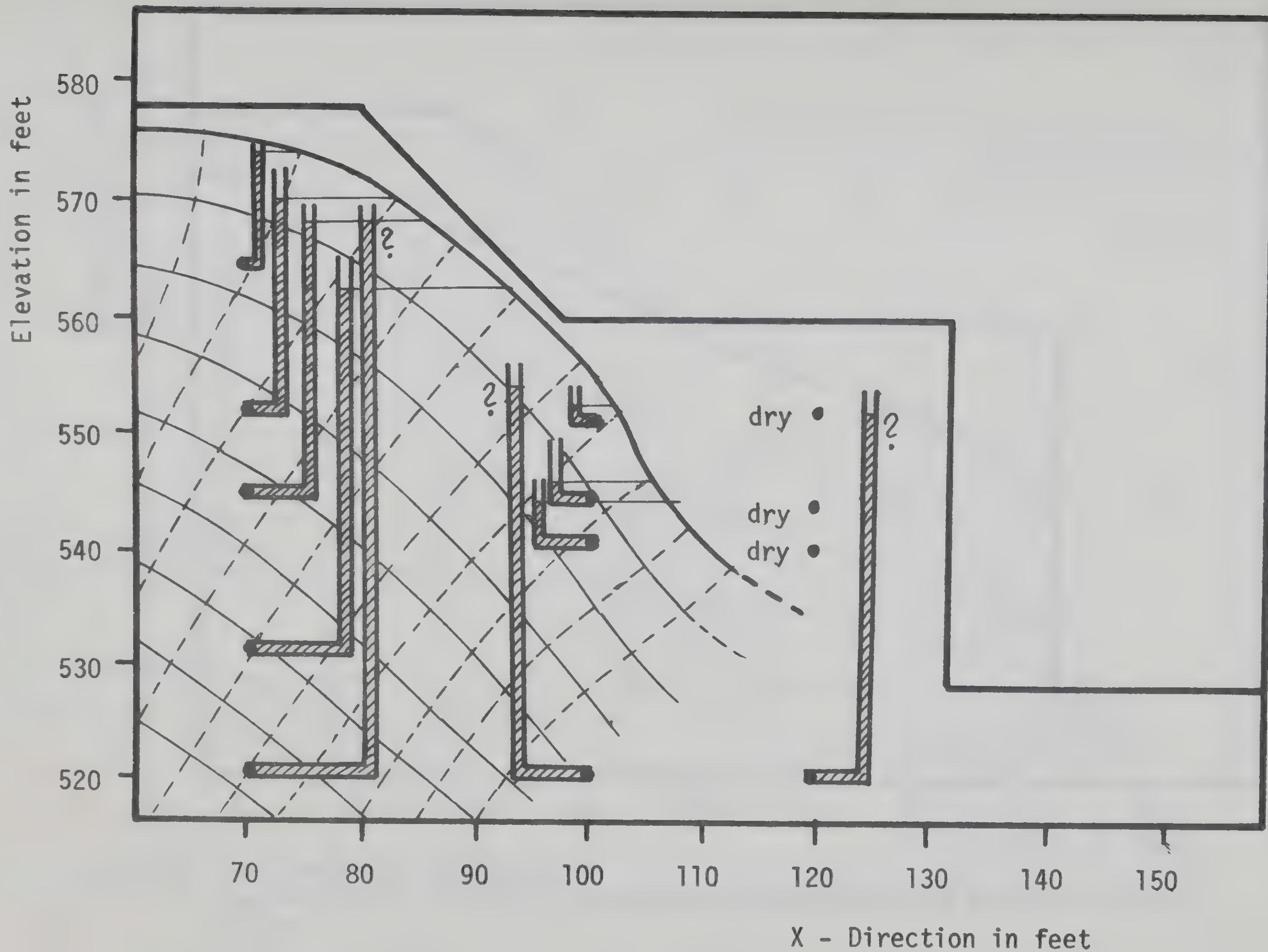


FIG. 5.24. WELLAND TEST PIT:
FLOW NET CONSTRUCTED ACCORDING TO THE PORE PRESSURE READINGS, MEASURED 5 DAYS AFTER END OF CONSTRUCTION OF TRIAL PIT (KWAN, 1971).

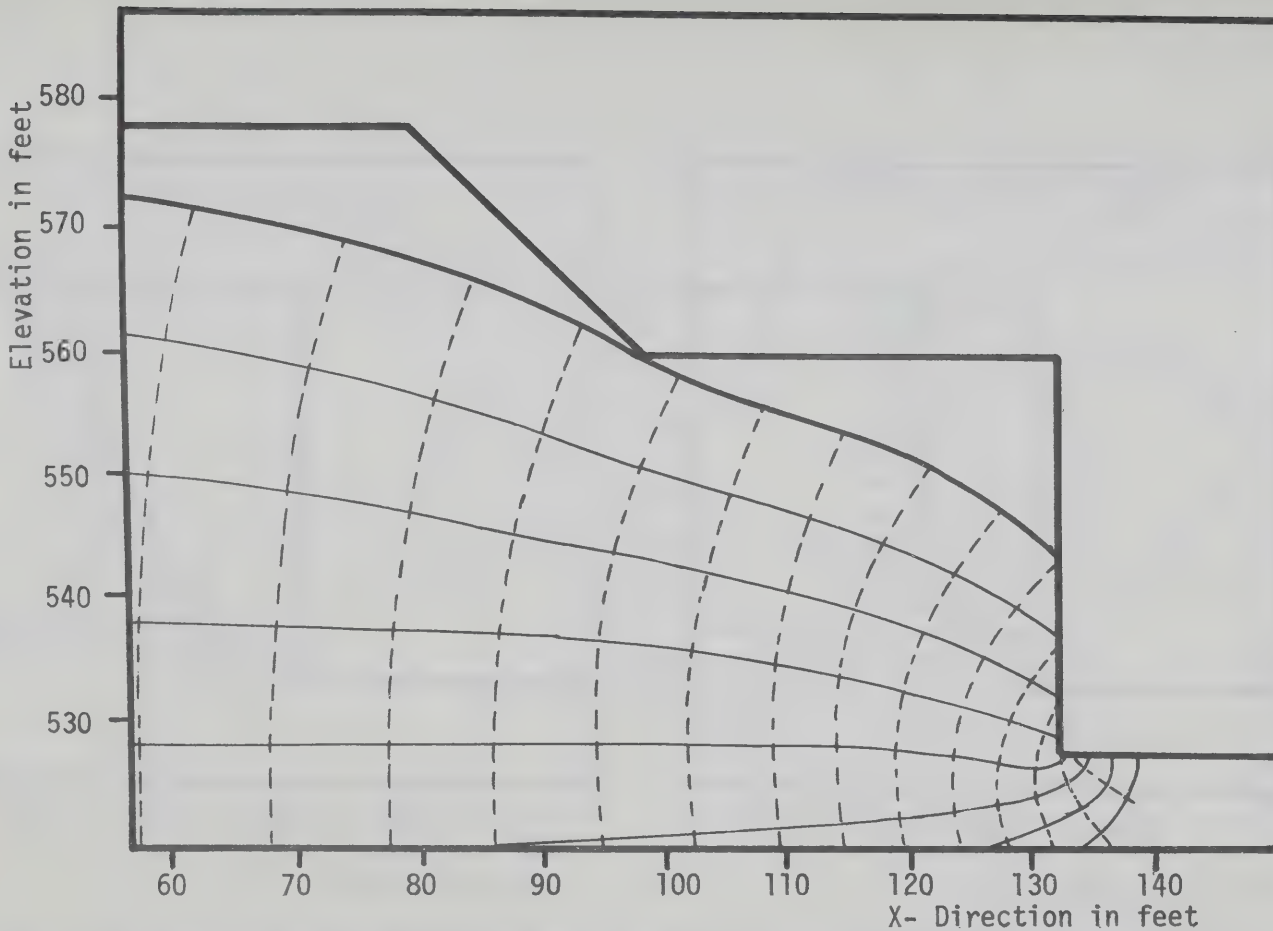


FIG. 5.25. PORE PRESSURE EQUALIZATION:
ASSUMPTION OF STABILIZED GROUNDWATER CONDITIONS FOR WELLAND TEST PIT.

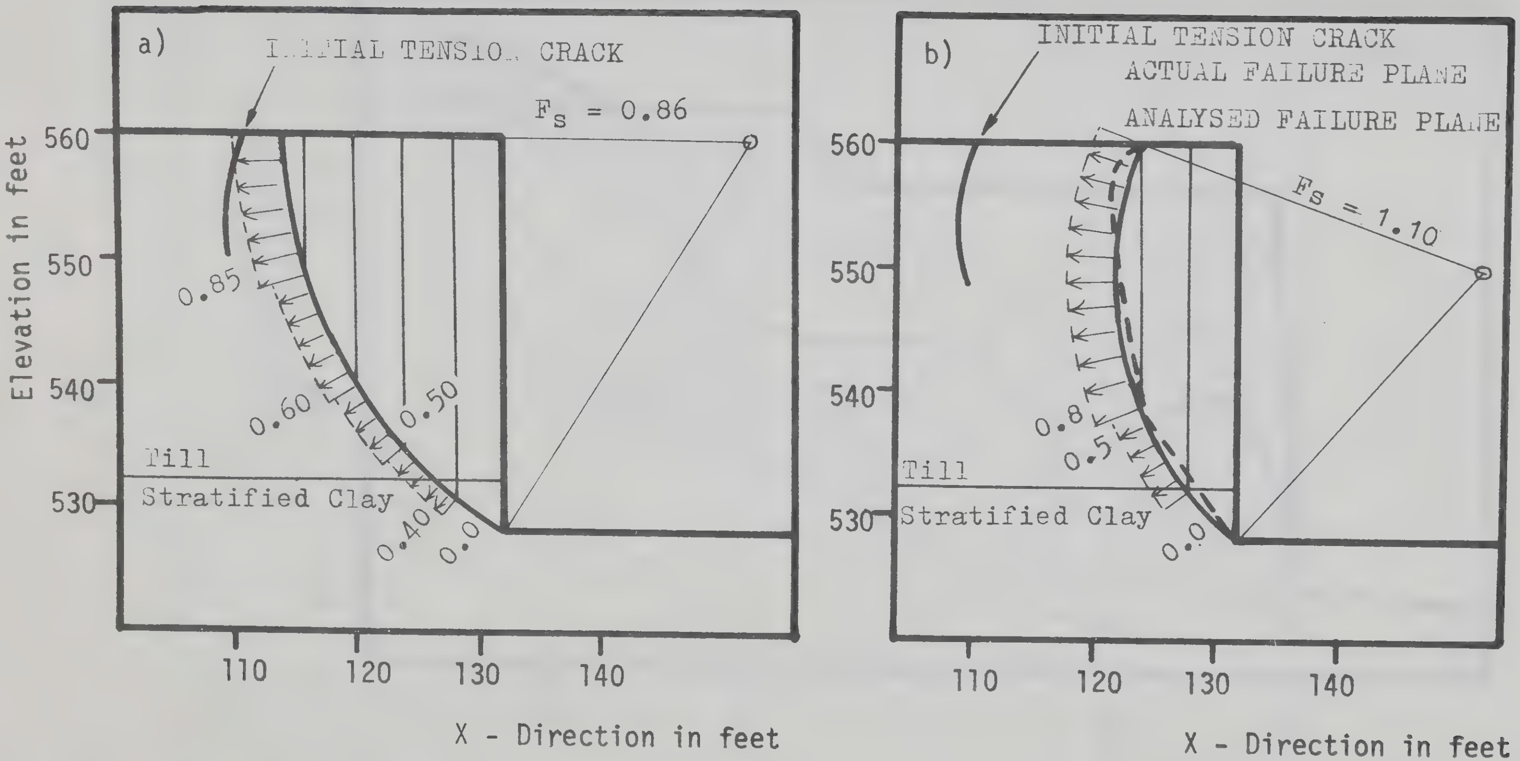


FIG. 5.26. STABILITY ANALYSIS AFTER BISHOP'S SIMPLIFIED EQUATION FOR WELLAND TEST PIT:
a) CIRCULAR ARE DESCRIBING MAXIMUM EXTENSION OF FAILURE SURFACE (SECTION a-a ON FIG. 5.17.)
b) CIRCULAR ARE DESCRIBING MINIMUM EXTENSION OF FAILURE SURFACE (SECTION b-b ON FIG. 5.17.).

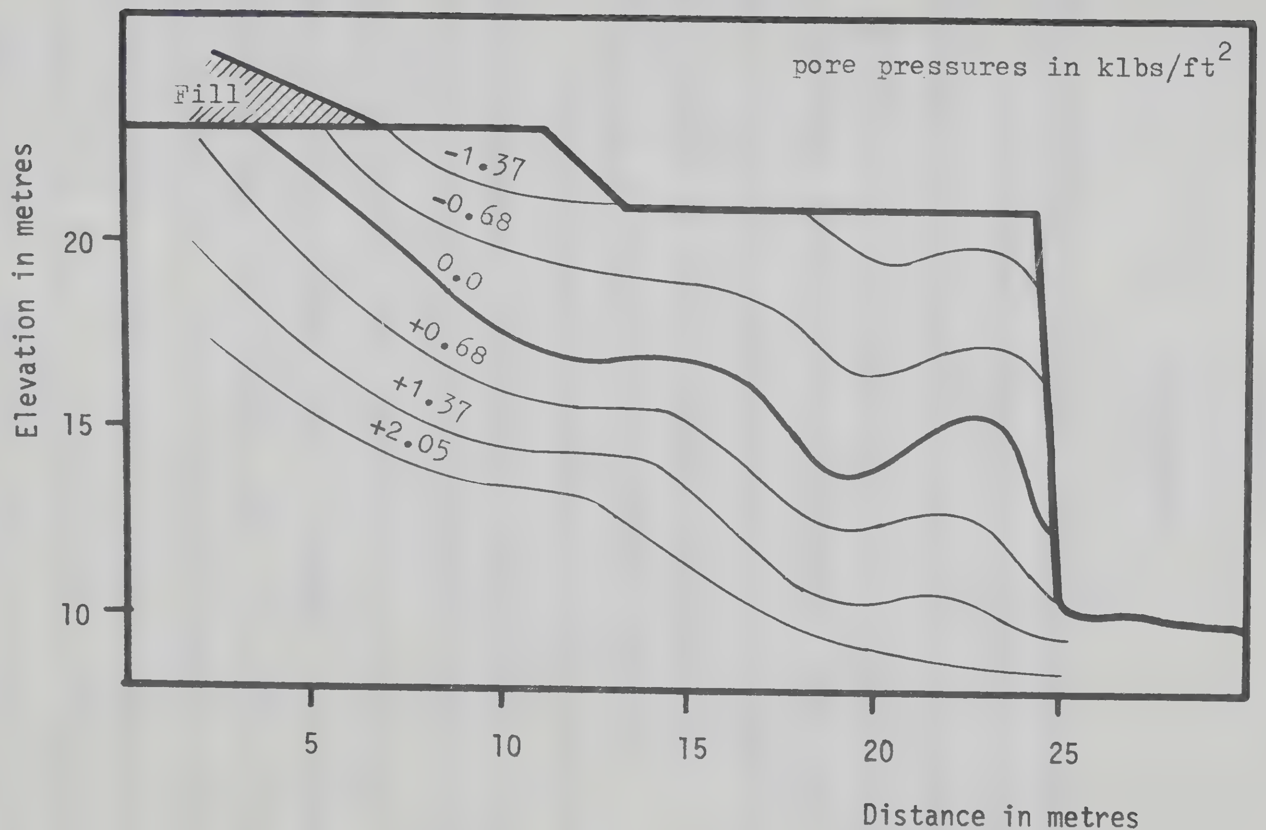


FIG. 5.27. PORE PRESSURES DUE TO UNLOADING:
MEASURED PORE PRESSURES IMMEDIATELY AFTER EXCAVATION OF CUT IN BOOM CLAY.

CHAPTER VI

CONCLUSIONS

1. Post Peak Behavior of Overconsolidated Clays and Mudstones

Field evidence indicates that progressive failure in overconsolidated clays and mudstones takes place in two successive stages. The first stage of progressive failure is characterized by the "fully softened state of strength" (Skempton, 1970) with $c' = 0$ and $\phi' = \phi'_{peak}$ as the governing parameters. The second and final stage is defined by residual strength conditions with $c' = 0$ and $\phi' = \phi'_{residual}$ as the governing strength parameters.

The first stage of progressive failure can be reached through softening processes as described by Terzaghi (1936) and Skempton (1948 and 1970) and through weathering. The second stage of progressive failure can be reached only after very large strains and after passing through the "fully softened state of strength" (e.g. Skempton, 1970; Schofield and Wroth, 1968; Roscoe, 1967). The post peak behavior in overconsolidated clays and mudstones can be compared with the post peak behavior of granular materials which have been investigated in much detail (Wroth, 1958; Roscoe, Schofield and Wroth, 1958; Roscoe, 1965; Rowe, 1965 and 1969). It has been shown that there is a remarkable similarity between the behavior of clays and cohesionless granular media

(Roscoe, Schofield and Wroth, 1958; Rowe, 1962; Barden, 1972). Similarly to granular materials, overconsolidated clays and mudstones start to dilate at greater shear deformations. This is indicated by an opening of fissures and an increase of water content along the zone of failure. At the same time the strength starts to decrease until it reaches the strength of a normally consolidated soil. It can be imagined that the softened shear zones contain numerous discontinuous shears (Skempton, 1970), which develop due to locally generated differential movements. Along these small shear zones orientation of the clay particles can be observed (Morgenstern and Tschalenko, 1967) and locally residual strength conditions might hold. The overall strength, however, is represented by the fully softened strength. During further displacements the number and extension of shear zones increases until some of them eventually link together and form one or several continuous shear planes, which initially show undulating surfaces. Finally, after very large displacements, a smooth main slip plane might develop along which the overall shearing resistance is determined by the residual strength conditions. Examples indicating the need of very large strains for residual strength conditions were reported by James (1970). Test results from the ring shear apparatus (Hvorslev, 1960; Bishop et al., 1971) also indicated the importance of very large strains for the reorientation of clay particles and the resultant reduction of angle of friction. Dilatancy and breakdown of cementation bonds are believed to result in a destruction of cohesion.

2. The Water Deterioration Tests as Indications of possible Strength Reductions

The standard compression softening test measures the loss in strength for undisturbed materials exposed to water at zero stress. The quantitative slaking test indicates the deterioration of disturbed materials, disturbed by cyclic drying and wetting. The rate of slaking test predicts the rate of deterioration during one cycle of drying and wetting.

Strength softening, as described by the compression softening tests, results in a reduction in strength as well as in stiffness. The decreased stiffness might lead to deformations at unchanged total stresses, as indicated by Fig. 3.12. These deformations might cause differential strains along differently softened zones. Joints might open up and slickensides might develop, thus disturbing the original structure of the material. Slight disturbances have been observed to initiate slaking (Balasubramonian, 1972; Hamon and Post, 1969; Nakano, 1967; Underwood, 1961). In these disturbed regions, slaking starts to take place and continues to weaken the material progressively at a faster rate than during strength softening. Fast slaking materials were found more sensitive to disturbances than slowly slaking ones. Accelerated weakening can be expected to occur sooner and more severely in fast and highly slaking materials than in less slaking materials. Softening is thought to occur in rock slopes as well. Because of the slower rate of reduction in stiffness and a lower degree of strength reduction during softening and

slaking, the process is far less severe and occurs at a much slower rate than in stiff and hard clays. However very thin soft zones along joint surfaces can create substantial strength reductions along these discontinuities (Rengers, 1971), and thus can influence the average strength in a slope material. A good example to illustrate the result of the combined action of strength softening and slaking in a hard clay is given by the back slope material of the Devon Slide (Eigenbrod and Morgenstern, 1971), where blocks of essentially unweathered materials are found separated by softened clay. Large scale triaxial tests revealed a substantially lower strength for the overall weathered material (including lumps of unweathered clay) than for the intact unweathered clay. A substantially lower cohesion intercept was found.

The influence of slaking is most spectacular in surface zones which are affected by cyclic drying and wetting. But slaking can also occur within zones of fluctuating water tables and vapor pressure (Nakano, 1967), or in zones which have been mechanically disturbed by tectonic movements (Nakano, 1967; Esu, 1967), e.g. in fault zones, or in zones of old landslides.

During slaking, substantial volume changes have been observed. These are very likely nonhomogeneous within the clay mass, and result in differential strains along zones of different volume change characteristics. Slickensides might be formed and as a consequence the effective strength might be reduced. Similarly, but at a larger scale, differential movements might occur along the interface of highly swelling and low

swelling materials (e.g. along the interbed between bentonite and coal), within slopes affected by slaking. Because of the very large possible volume changes, these differential strains can be imagined as high enough to shear the soil to such a degree that not only the cohesion intercept, but also the angle of friction might be reduced. This process however, is probably very slow, so that it might be very difficult to substantiate it by field evidence.

During slaking the Liquidity Index of a clay or mudstone increases. Eventually a very soft state can be reached described by a Liquidity Index $I_L = 1$. Extensive slaking, which is mainly restricted to surface zones, leads often to mudflows in the weakened surface zones of clays or mudstones (Esu, 1967). However, sometimes temporary increase of strength during a slaking process can also occur, when materials, such as ironoxides or carbonates, penetrate into the slaking materials and act as cementing agents.

3. Field Evidence of Strength Reduction

A general correlation between landsliding and water deterioration has been reported by Nakano (1967) for Japanese mudstones. Locker (1969) observed for the bedrock in Central Alberta, Canada, that the Liquid Limit decreases from East to West. High Liquid Limits were found to indicate highly slaking materials. Accordingly the number of landslides along the river valleys decreases in a westerly direction. A study of river valleys in Alberta revealed a close correlation between landslide

activity and lithology (Roggensack, 1971; Arvidson, 1972). From other areas there is ample field evidence for relatively stable slopes in little and slowly slaking clays: e.g. in Ball Clay (Best and Fookes, 1970), or in kaolinitic sediments in southern Alberta. The observed correlation between number of landslides and water deterioration properties of the materials involved indicates the importance of strength reduction resulting from dilatancy and increase in water content.

Fully softened strength conditions as reflected in $c' = 0$ and $\phi' = \phi'_{peak}$ were measured for slaked materials. Hamon and Post (1969) reported for shale from Java after slaking in the field the same strength parameters as after remoulding. Shear tests on Cucaracha Shale from the Panama Canal were performed after the samples had been slightly dried and consequently wetted to equilibrium water content. Fully softened strength conditions ($c' = 0$ and $\phi' = \phi'_{peak}$) were found to govern the strength of these samples. Fully softened strength values together with representative pore pressure values explained the failures at the Panama Canal (USCE, 1971). The partly softened strength results obtained for the back slope materials of the Devon Slide (Eigenbrod and Morgenstern, 1971) were supported by the stability analysis of the slope failure. Softened strength values were similarly considered along the back slope materials of the Lesueur Slide (Thomson, 1971) and a more satisfying explanation for the slope failure than previously given could be obtained. A large number of cases of first-time slides were quoted by James (1970) in which softened strength parameters had been mobilized at failure.

The possibility of residual strength conditions mobilized along a first-time failure plane cannot be conclusively supported by field evidence. Bishop (1971), however, pointed out that a nonuniform mobilization of strength from peak to residual along the slip surface might also account for known cases of first-time slides. A large number of reports on stable slopes in materials with unstable stress strain curves indicate that there is no mechanism which can create residual strength conditions in a slope before failure (e.g. Esu, 1967; Kankare, 1968; De Beer, 1969). For the slopes of the upper Kimola Canal (Kankare, 1968) the Factor of Safety in terms of peak strength was equal to 1.16 and in terms of fully softened strength less than one, at the most critical groundwater conditions. However, no failure has occurred yet.

The possibility of nonhomogeneous strength mobilization in slopes appears to be reduced due to the effect of yielding. Yielding in natural slopes causes stress redistribution: Unloading of the yielded zone occurs and subsequent increase of stress in zones which have not yielded so far. This process could be recognized in a Finite Element model of a slope for which an unstable stress strain curve and yielding was considered (Gates and Zehrbach, 1972). The stress redistribution consequent to local yielding in a slope tends to reduce initially induced stress concentrations. This process counteracts the tendency to form a localized zone of failure, as observed in triaxial test specimens. Nonhomogeneously mobilized strength along a potential failure plane be-

[illegible]

comes more unlikely. The likelihood of the nonhomogeneous mobilization of strength is the greatest along interfaces of soft and hard materials. This, however, is difficult to prove by field evidence, since these zones are often already sheared in place. Further it must be remembered that only average strength values can be considered in a failure analysis.

Case histories of slides in slightly overconsolidated non-fissured clay with flat-topped stress strain curves suggest that these materials are not prone to any kind of progressive failure.

Summarizing the results of the investigations it can be suggested that the design of permanent slopes which have not failed previously can be based on the fully softened strength parameters:

$c' = 0$ and

$\phi' = \phi'_{peak}$ - as measured from representative samples.

Slopes in non-fissured clays with a Work Softening Index less than 0.2 can be designed on the basis of peak strength parameters.

4. Importance of Zones of Nonhomogeneities

Most natural slopes are nonhomogeneous in terms of strength as well as in terms of water pressures. This is indicated for the majority of the slides investigated. Zones of low strength are often small features and therefore very difficult to detect. However, the mechanisms, which created weak zones, often can be easily recognized, such as old landslides, glacial ice movements, or tectonic movements. Whenever these conditions are prevalent failure zones must be expected. The presence of

sheared clay in place, however, is not always indicated by dramatic geomorphological evidence. Less pronounced features, such as anticlinal rebound of strata in response to valley formation, nonuniform swelling, solifluction, or sedimentological features, can also account for failure zones. Therefore a special concern for minor geological details is always warranted.

To properly evaluate the stability of a natural slope, the true water pressure distribution has to be known. Particularly in complex sedimentary bedrock, as demonstrated by the Devon Slide (Eigenbrod and Morgenstern, 1971), detailed studies are needed to detect the representative pore pressure distribution. Nonhomogeneous water pressure distribution is not only dependent on the local geology but can also be caused by stress induced permeability changes during excavation of a slope (Guther, 1972). Time-dependent pore pressure changes, such as dissipation of negative excess pore pressures, or pore pressure changes correlated to climatological conditions, affect the stability of slopes. Only adequate piezometer installations, which take these factors into account, can provide representative results.

5. Analyses of the Pore Pressure Changes following the Excavation of a Slope

In a numerical analysis the pore pressure changes due to excavation of a slope were calculated. The analytical results agree reasonably well with pore pressure measurements in comparable slopes. This

suggests that pore pressures immediately after slope excavation can be predicted analytically in homogeneous materials.

The results of an analysis dealing with the dissipation of excess pore pressures due to unloading can also be substantiated by field evidence however, very few comparable field data are available. The time for pore pressure dissipation is often in the range of several hundred years. Therefore many failures might be caused by the delayed equalization of pore pressures. For many slopes it can be noted that the time for full dissipation is of the same order of magnitude as the time between excavation and failure.

6. Classification of Noncalcareous, Inorganic, Sedimentary Materials

Overconsolidated clays and mudstones have been classified on the basis of water deterioration tests. The standard compression softening tests separate two groups of materials on the basis of strength loss during softening. The one group of materials never loses more than 50% of its strength and is called rock. The second group loses more than 50% of its strength during softening and is called clay. Clays can be further classified according to the rate of strength loss: Hard Clays lose 50% of their strength within a period of more than one day. Stiff Clays lose 50% of their strength within hours. The amount of strength loss was found to be predictible from the compression strength of the unsoftened material.

Clays and mudstones can be further characterized in terms of slaking properties. The maximum water content obtained during slaking

was equal to the Liquid Limit of the respective material. Therefore the Liquid Limit can be used in predicting the maximum amount of slaking which can be expected for a material. The rate of slaking is described by the change of Liquidity Index during water immersion of ovendried specimens. The rate of slaking test is very helpful in the interpretation of the standard compression softening test results. Data which deviate from the general trend can be explained, and data in the zone of transition between rocks and clays can be better evaluated. For example, Clagget Shale showed the initial compressive strength of a rock, but nevertheless was found to lose more than 50% of its strength within hours just like a stiff clay. This discrepancy can be explained by its exceptional high rate of slaking, which indicates that already slight disturbances would initiate slaking in Clagget Shale.

The water deterioration tests are believed to provide much quantitative information which is useful for most engineering projects dealing with overconsolidated clays and mudstones in the design of long-term excavations, embankments, dams, tunnels, and all kinds of mining operations.

7. Recommendations

The validity of the present study on progressive failure is limited by the small number of complete case histories of first-time slides. Therefore more detailed investigations of first-time failures are needed.

The present investigation is concerned only with overconsolidated clays and mudstones. It might be important to expand the study to other problematic materials, such as quick-clays.

The Finite Element Analysis dealing with pore pressure changes upon excavating of slopes, as well as the calculation of the dissipation of excess pore pressures due to unloading can be further systemized and refined. This, however, is justified only if more detailed comparable field data are available. There is a particular need for continuous pore pressure observations in fresh cuts, taken from the start of the excavation until steady seepage conditions are reached.

An analysis of the volume changes during the dissipation of pore pressures might be worthwhile in order to investigate the possibility of large differential strains between swelling and nonswelling layers. A comparison between an analytical model and a laboratory model could indicate the conditions which favour large differential volume changes, and could possibly show whether the resulting differential strains can in fact create continuous shear planes.

The water deterioration tests have been developed for a wide range of materials originating in various parts of the world. Nevertheless, it might be useful to find out whether the obtained correlations hold true for all noncalcareous, inorganic, sedimentary materials, such as tuff, or other kinds of materials which have not been considered in the present study. The test results are correlated

to the behavior in practice only in terms of Index Properties and qualitative information. It might be useful to summarize all available data and to show, possibly quantitatively, how the test results apply to specific engineering design problems.

A correlation with other classification tests might also be warranted.

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APPENDIX

APPENDIX A

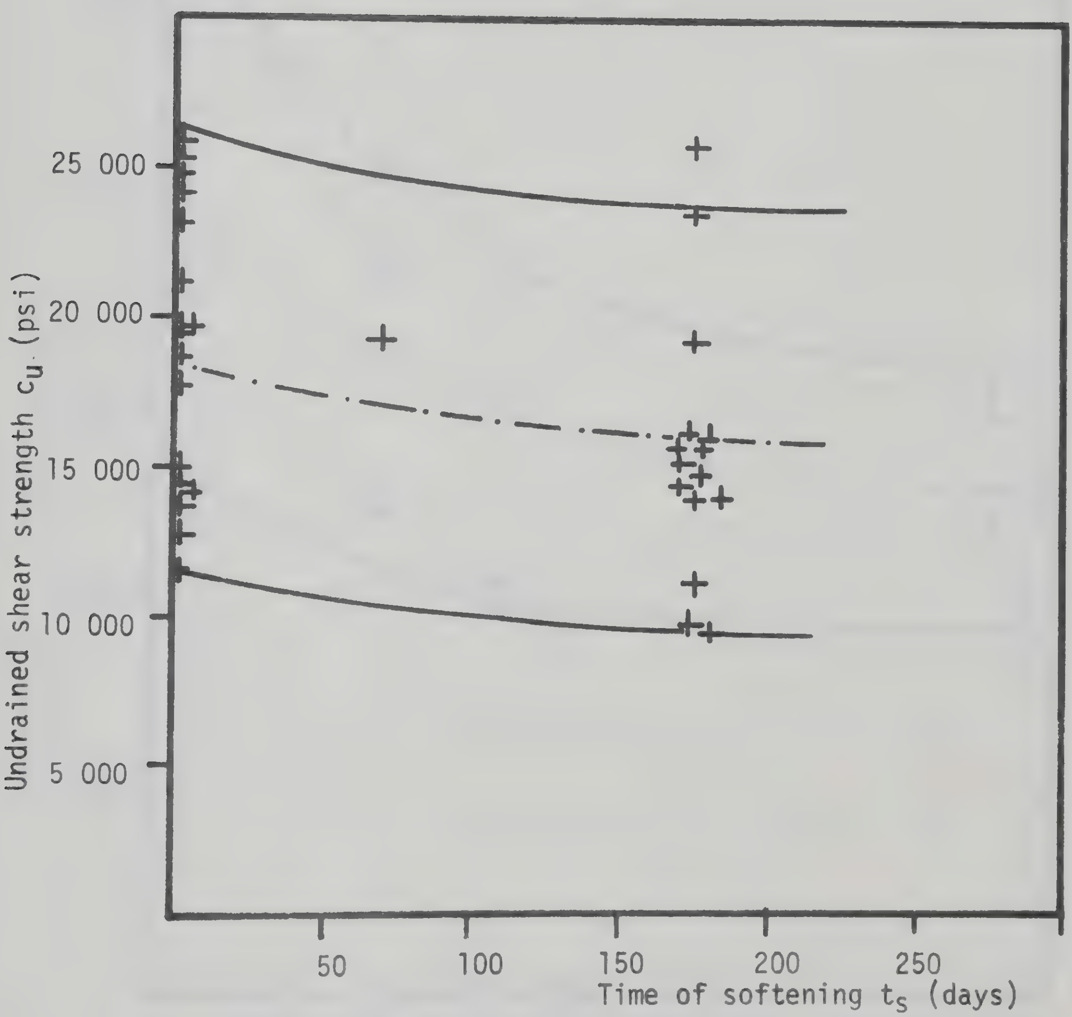
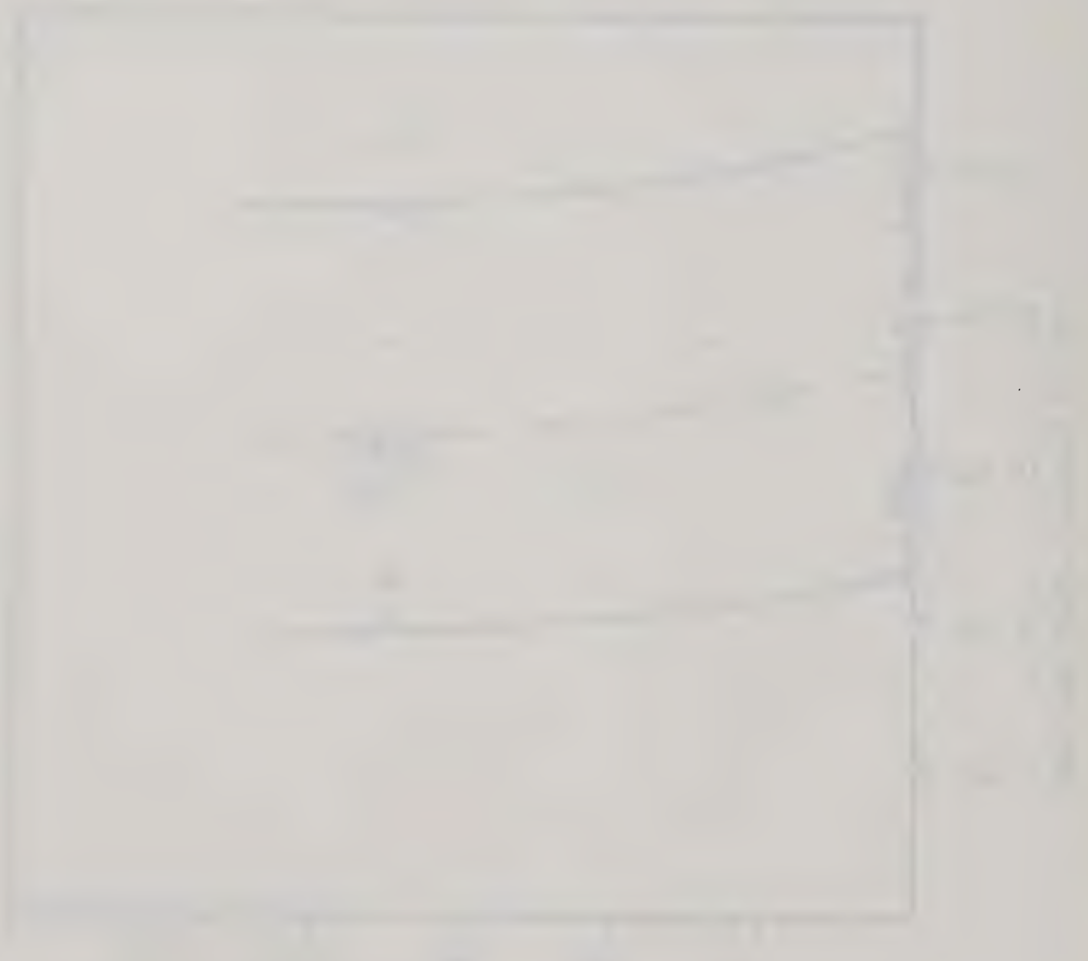


FIG. A 3.1. STANDARD COMPRESSION SOFTENING TEST: UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 1



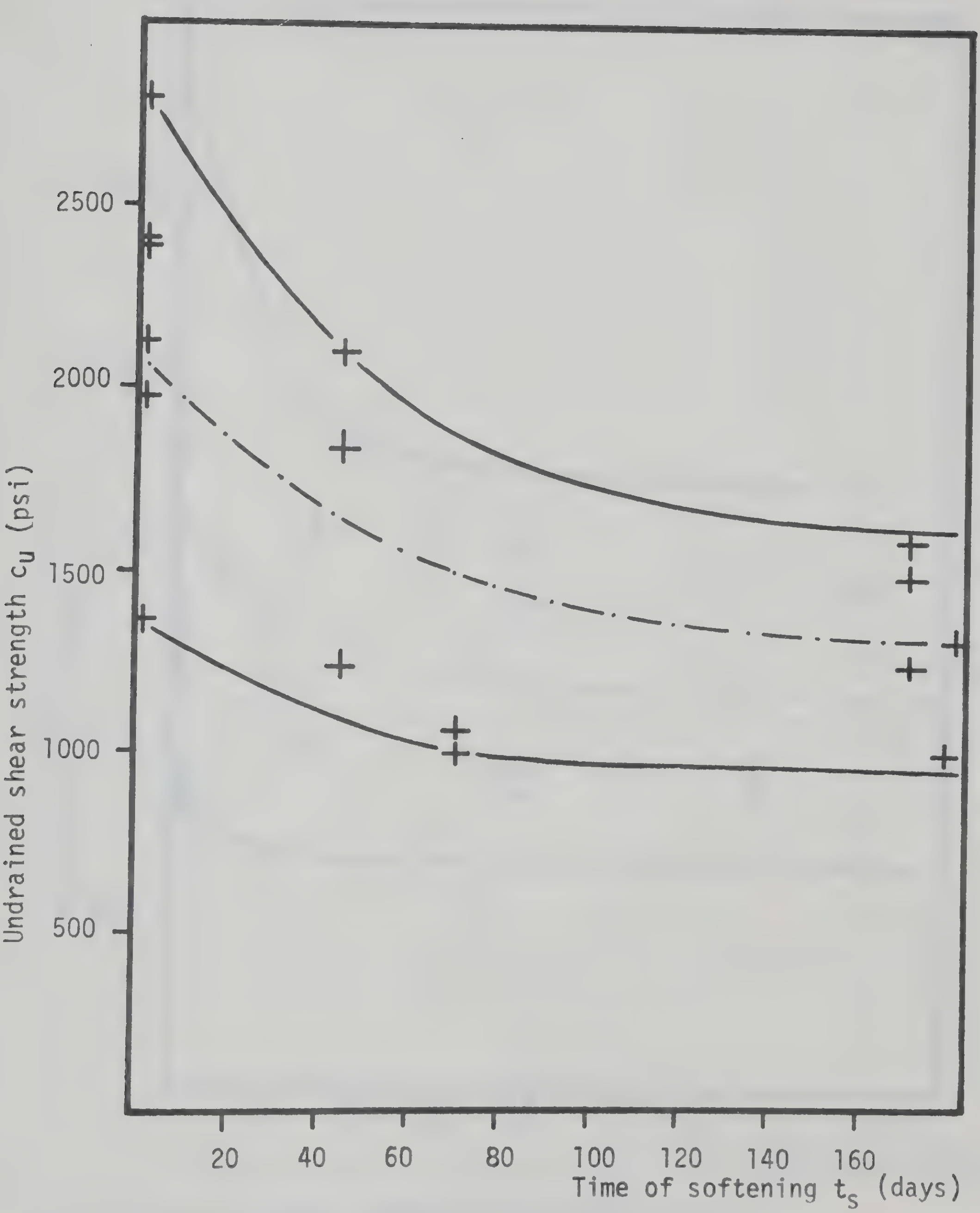


FIG. A 3.2. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 2

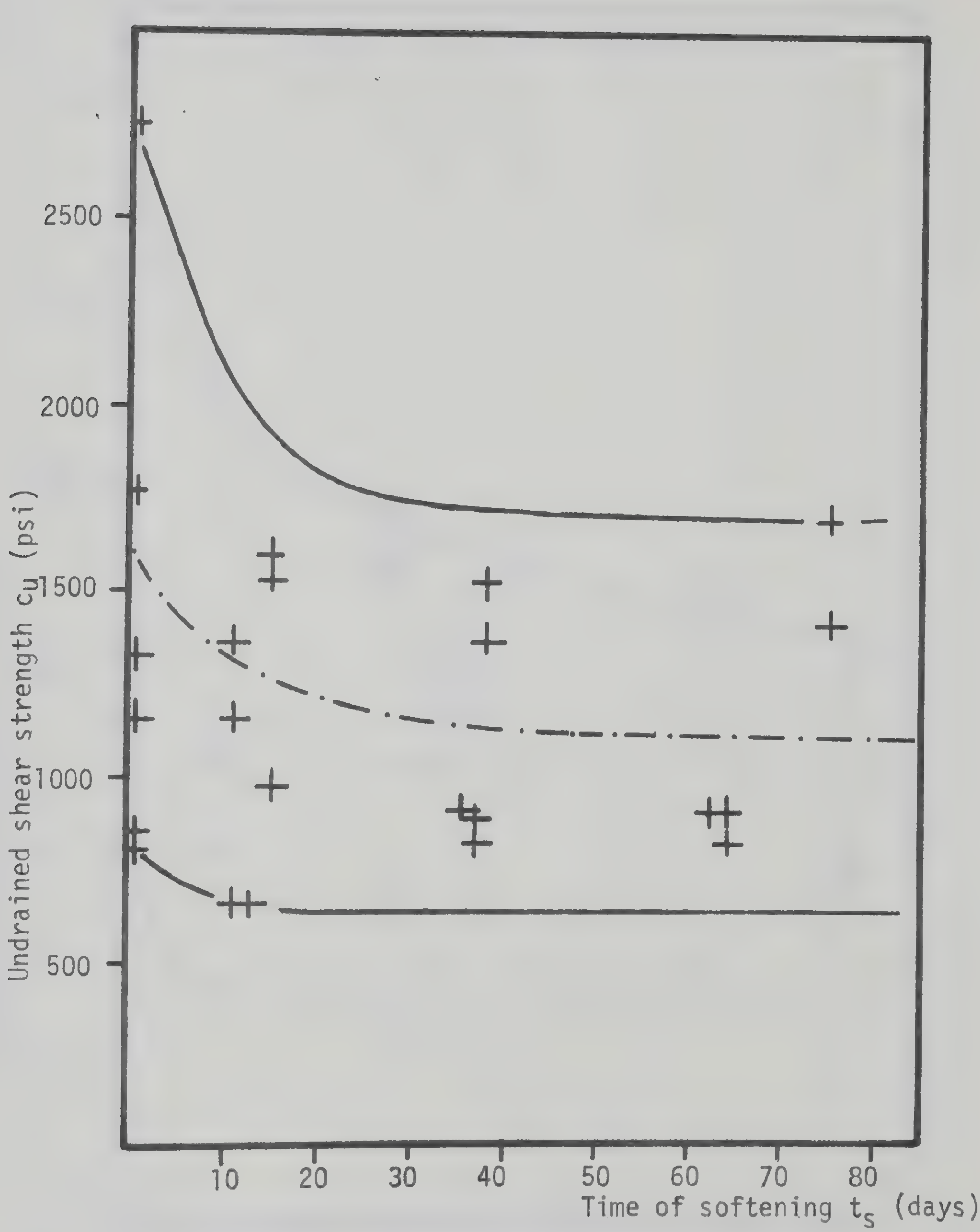


FIG. A 3.3. STANDARD COMPRESSION SOFTENING TEST: UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 3.

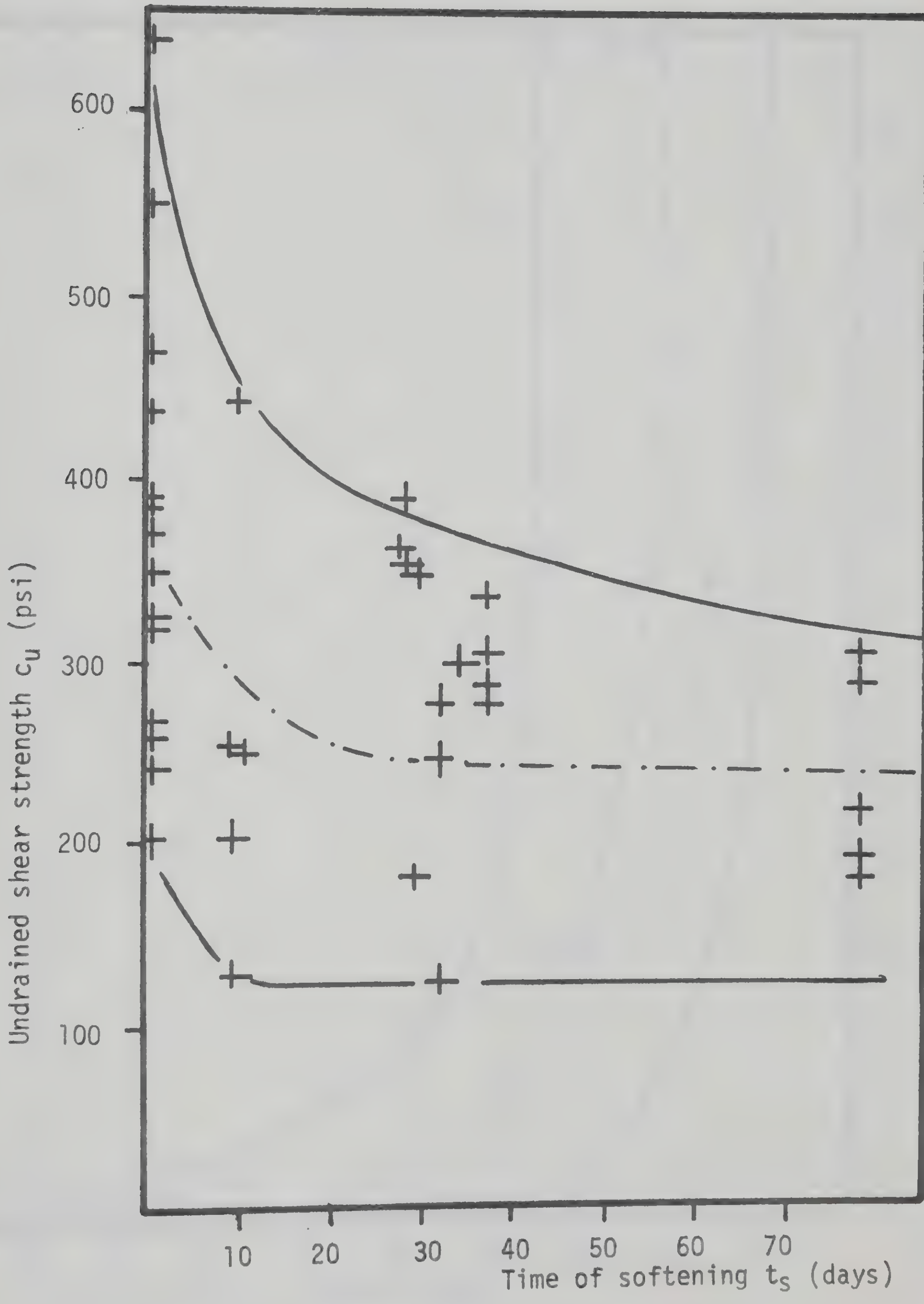


FIG. A 3.4. STANDARD COMPRESSION SOFTENING TEST: UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 4.

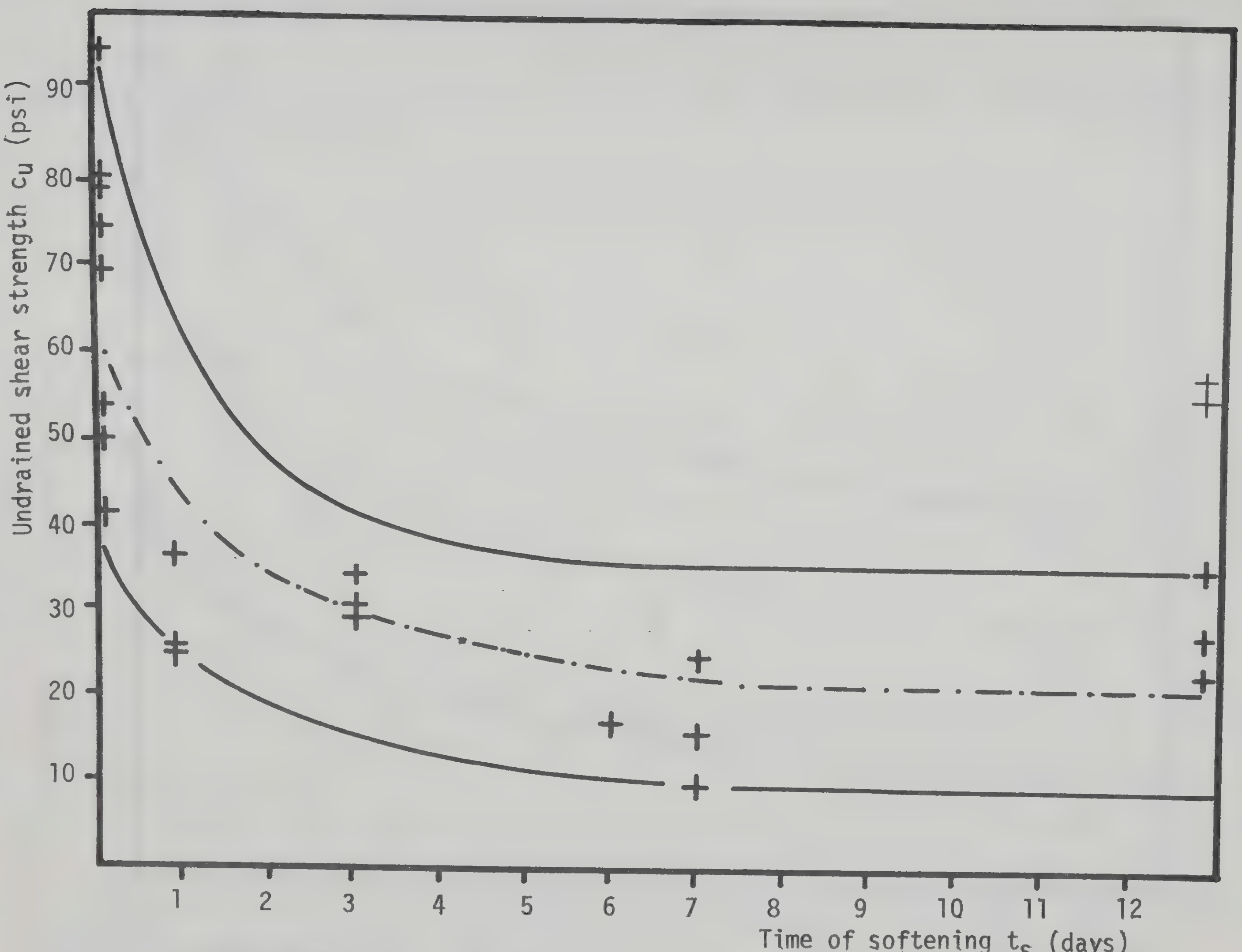
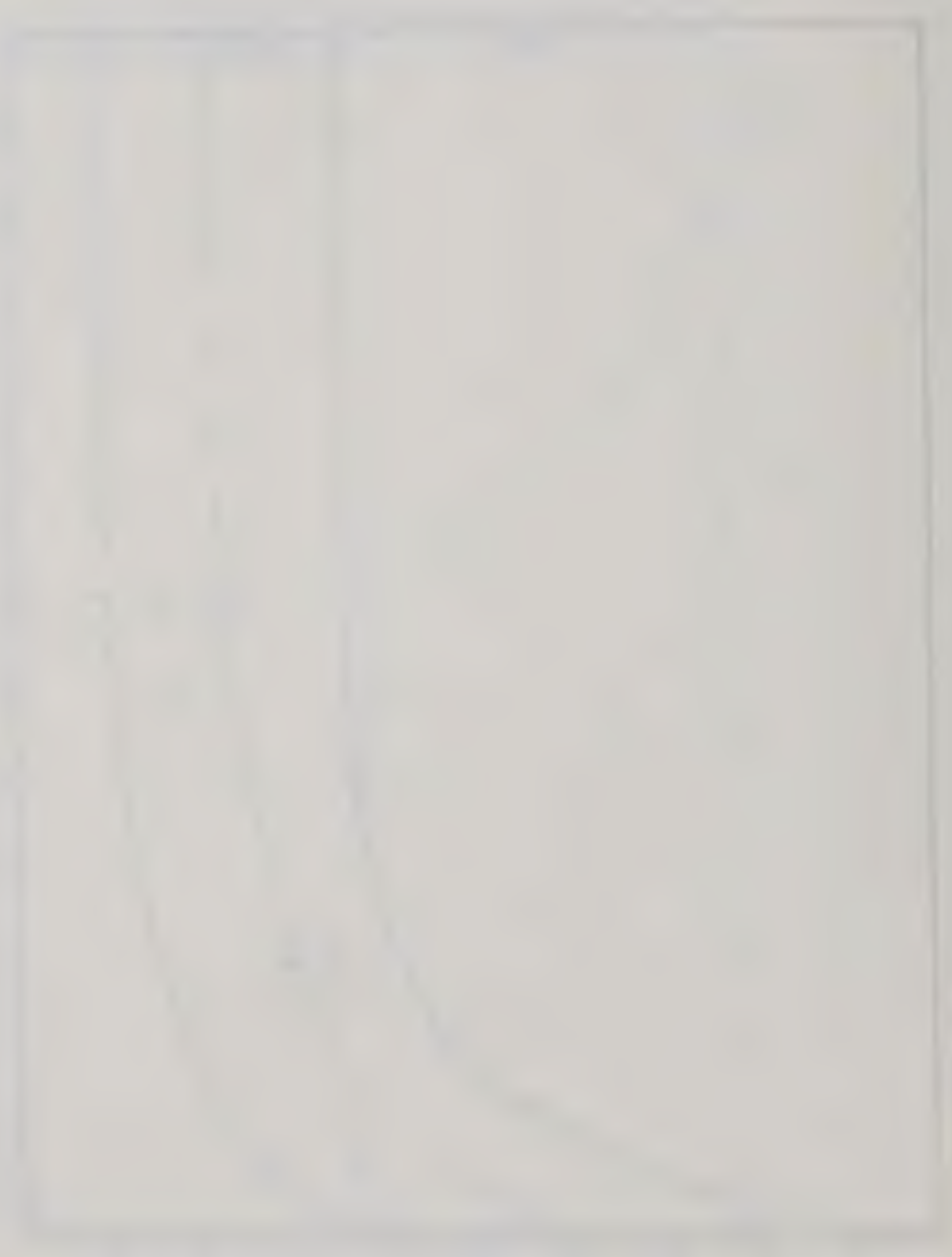


FIG. A 3.5. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 5.



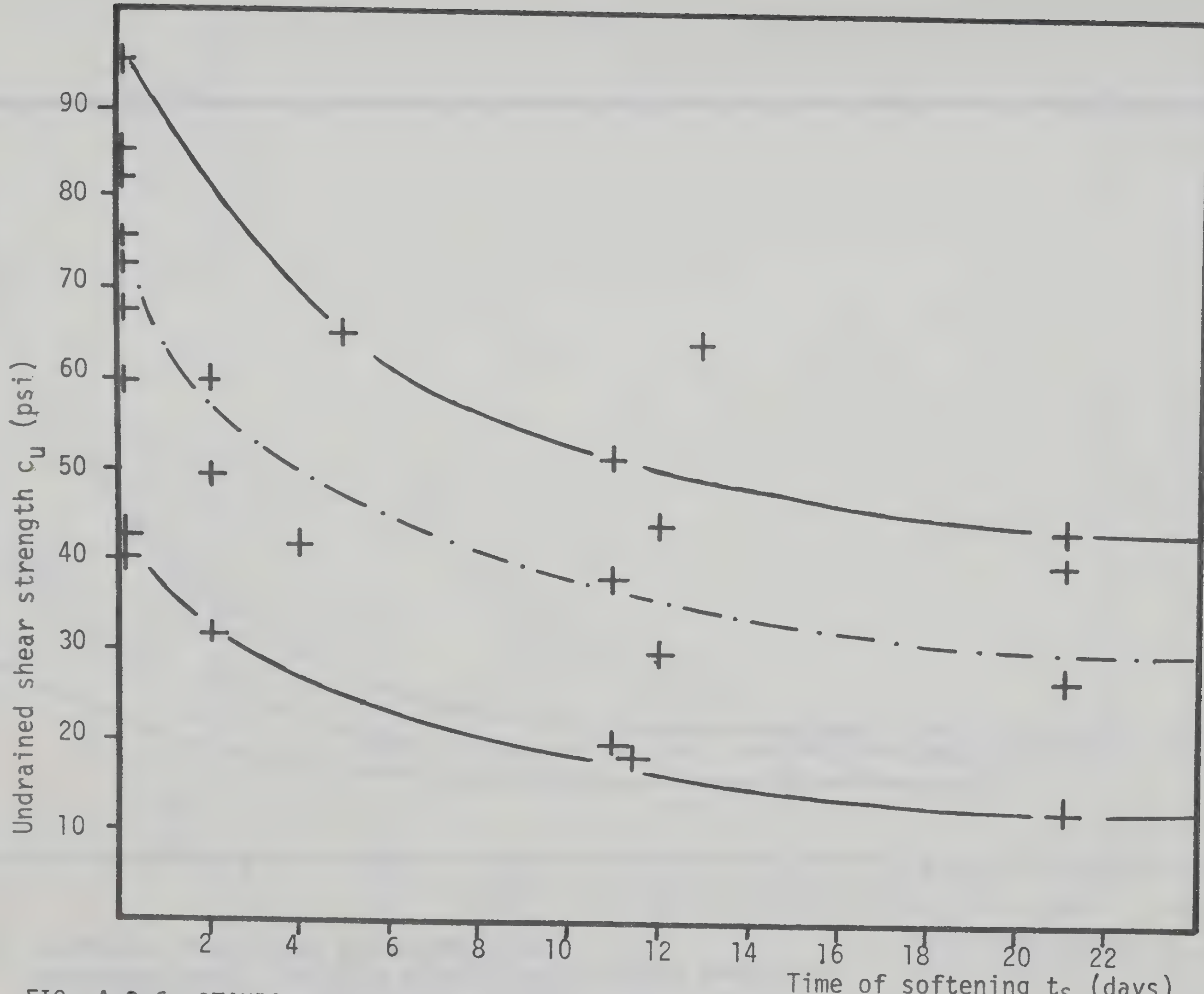


FIG. A 3.6. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 6.

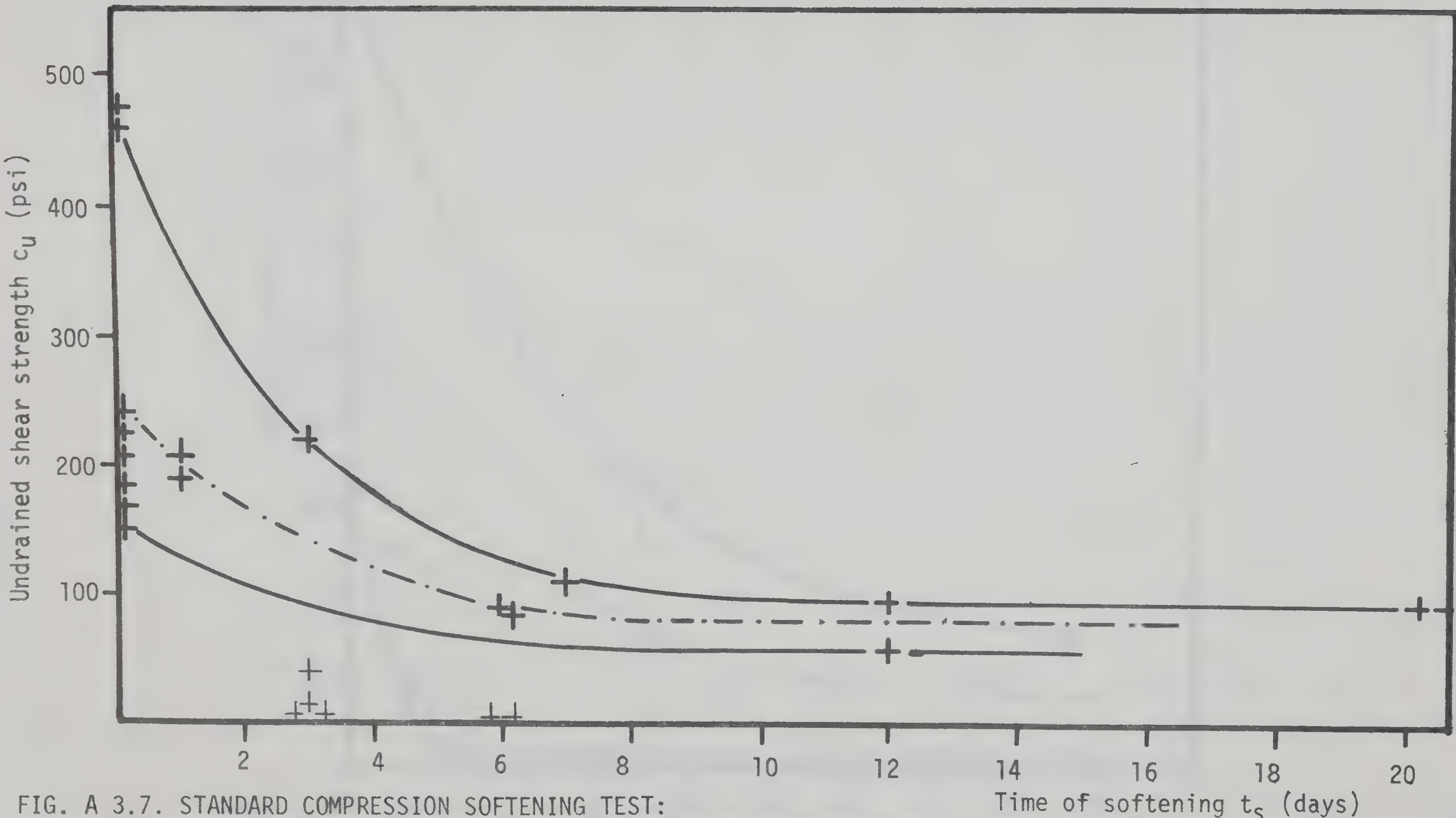


FIG. A 3.7. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION
FOR MATERIAL NO. 8 (BEARPAW SHALE).

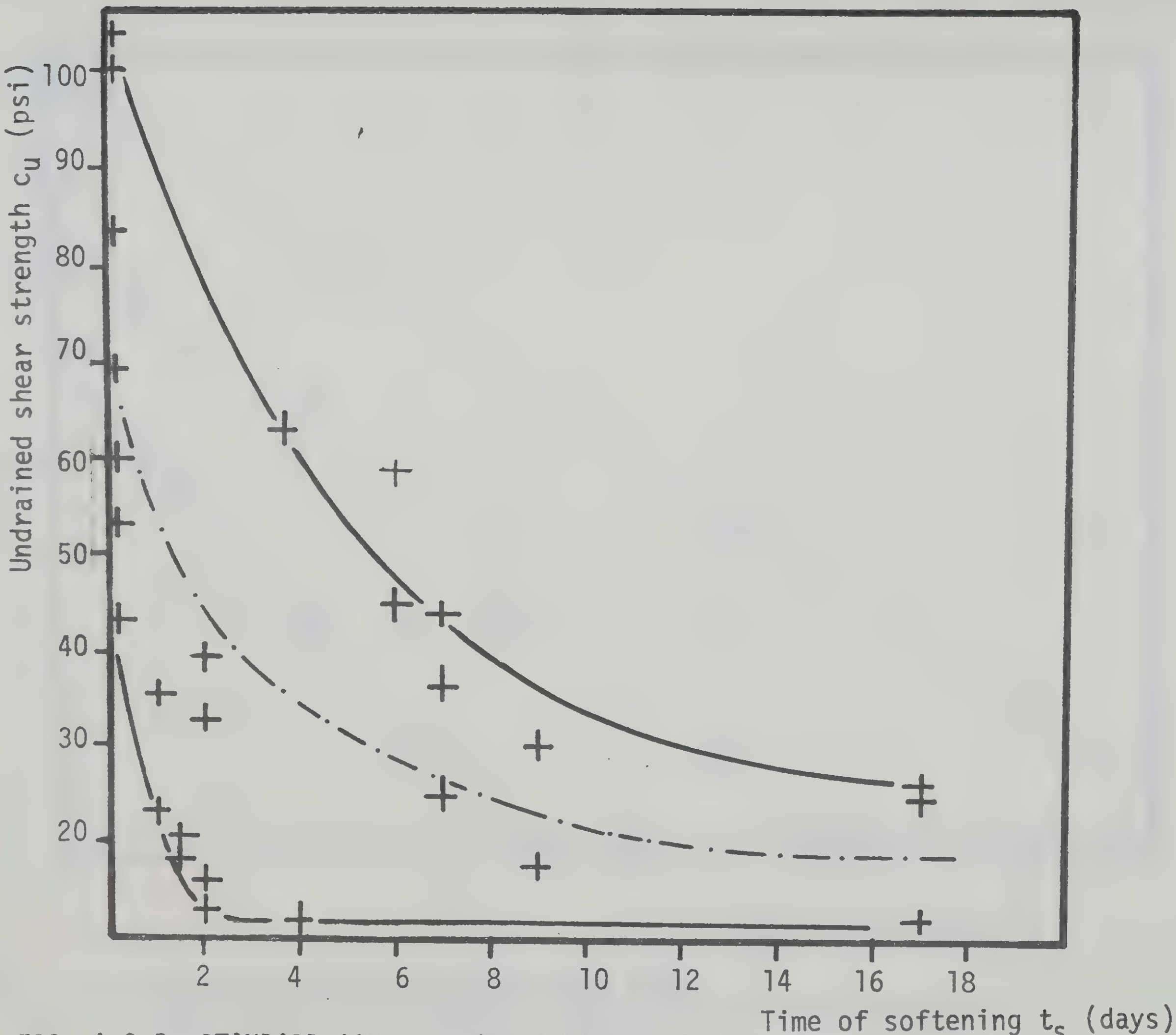


FIG. A 3.8. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 9
(OXFORD CLAY).

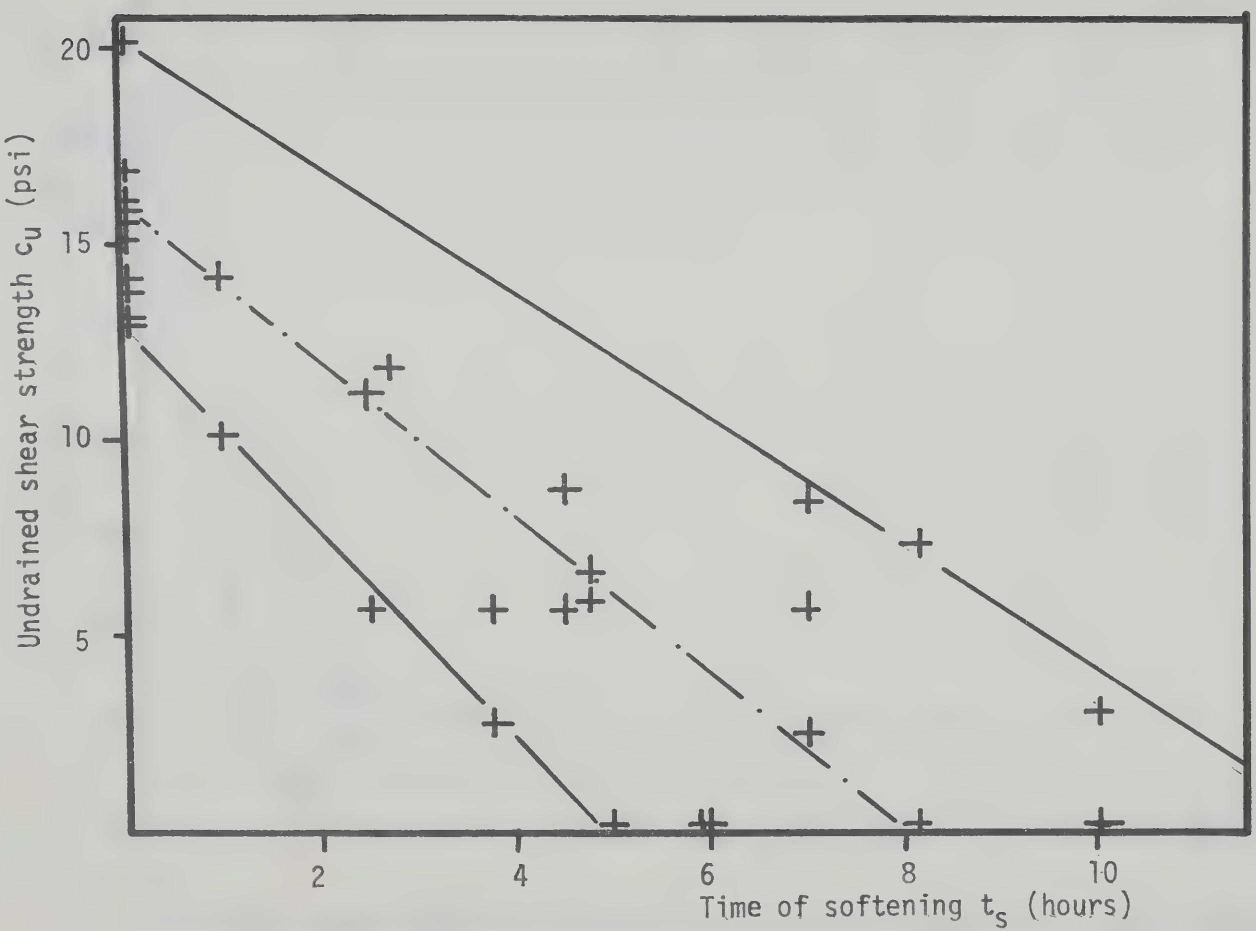


FIG. A 3.9. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION
FOR MATERIAL NO. 10 (GAULT CLAY).

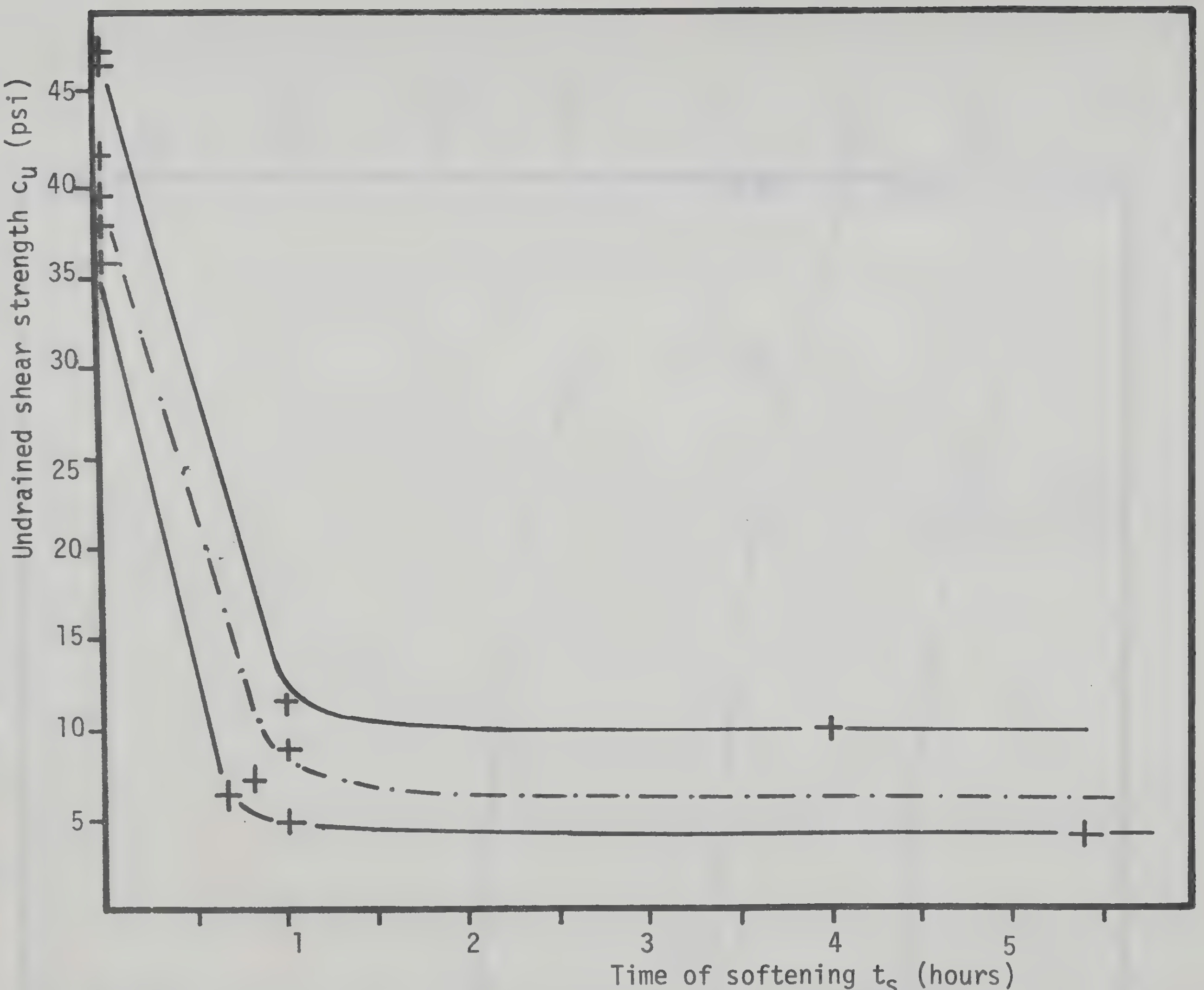


FIG. A 3.10. STANDARD COMPRESSION SOFTENING TEST: UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 11 (LONDON CLAY).

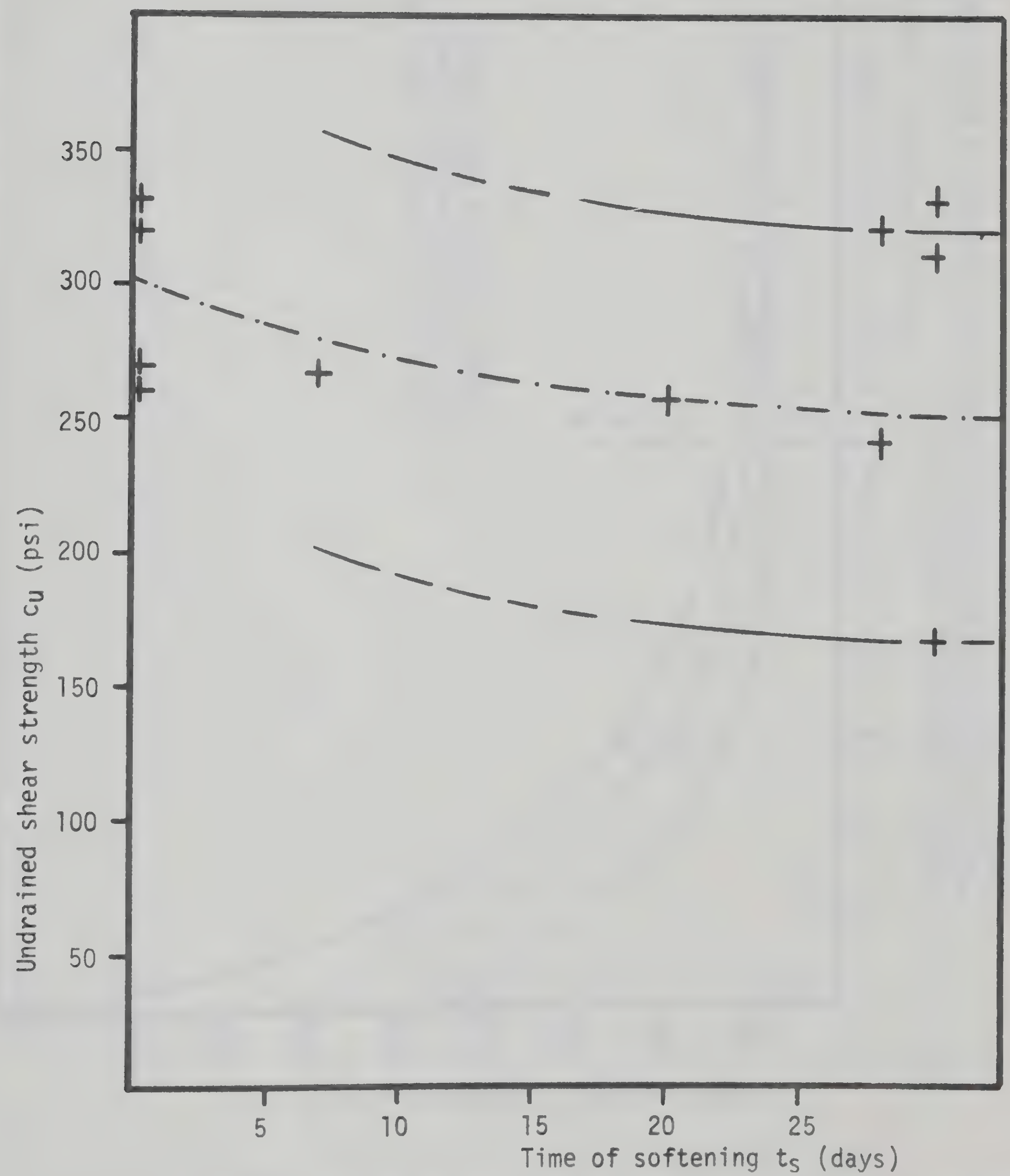
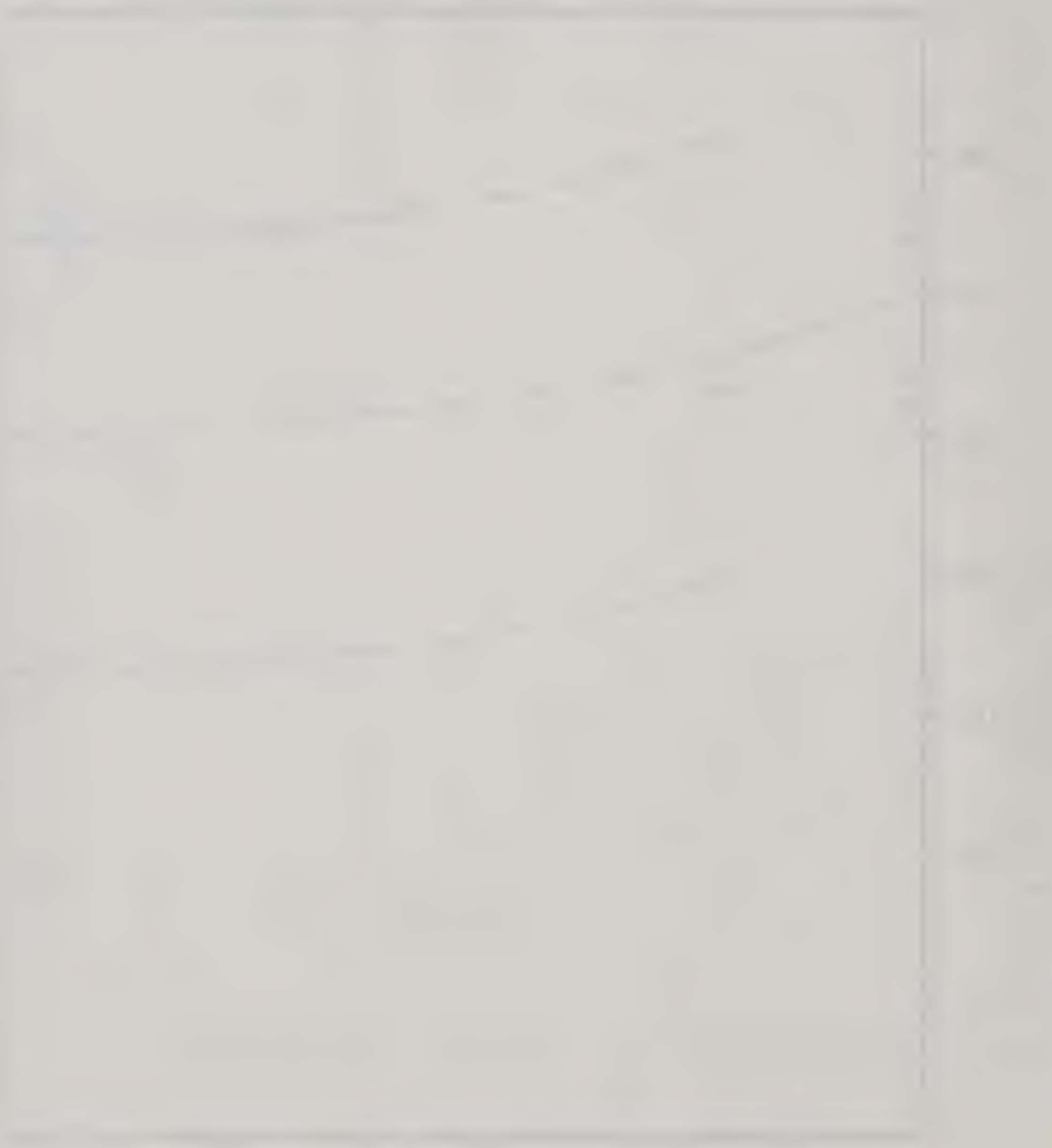


FIG. A 3.11. STANDARD COMPRESSION SOFTENING TEST: UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 12 (PIERRE SHALE).



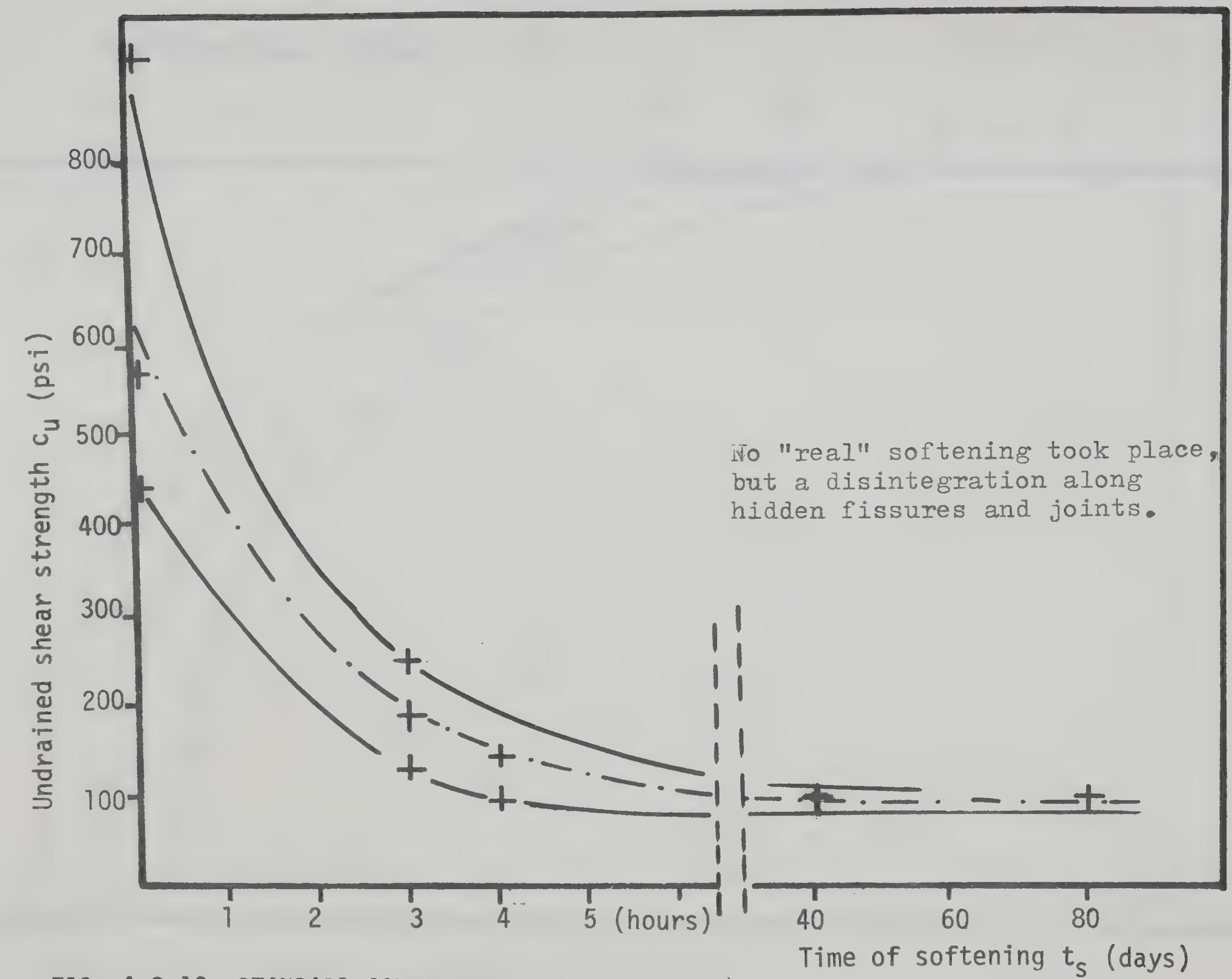


FIG. A 3.12. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION FOR MATERIAL NO. 13 (CLAGGET SHALE).

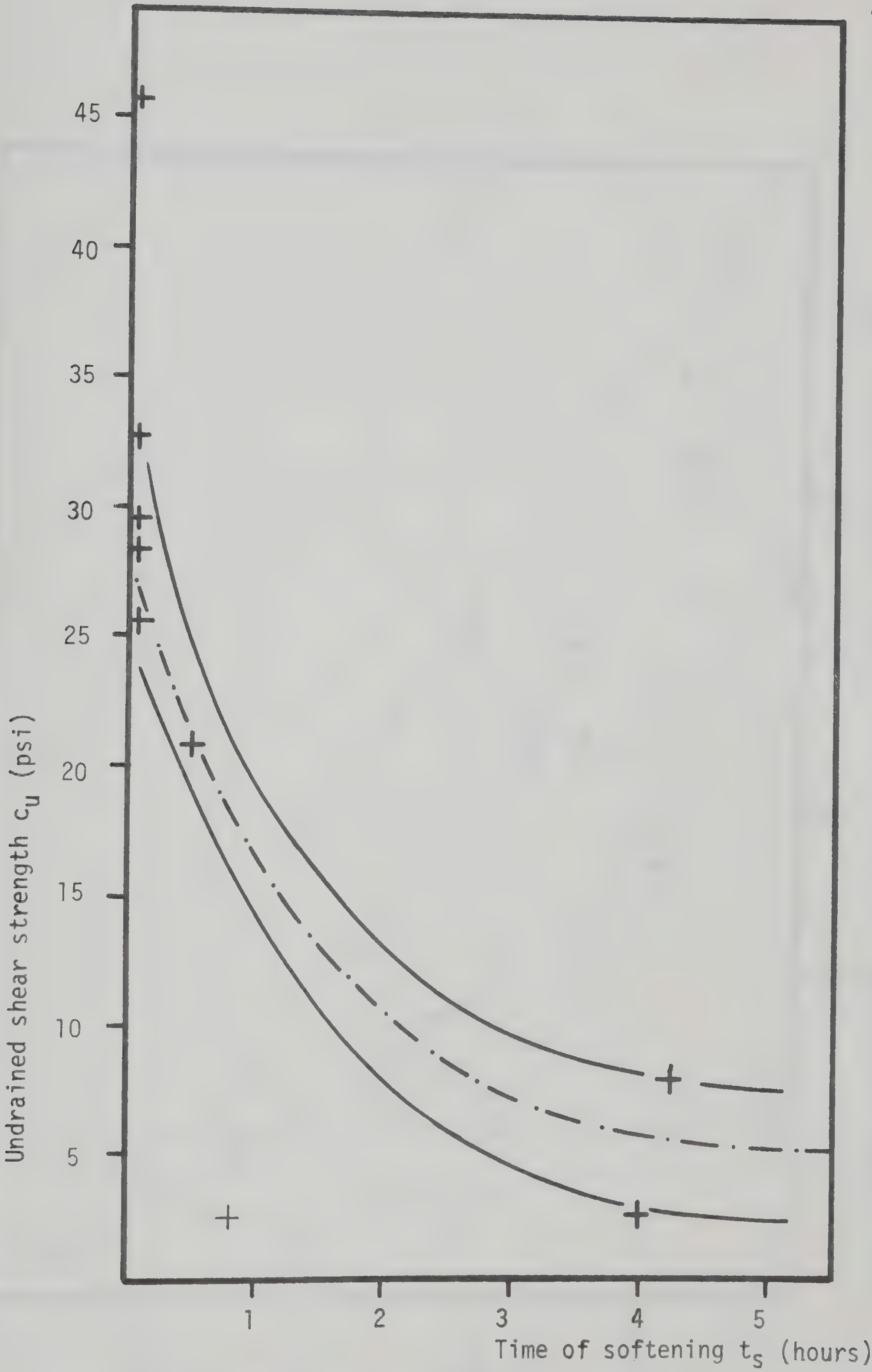


FIG. A 3.13. STANDARD COMPRESSION SOFTENING TEST:
UNDRAINED SHEAR STRENGTH VERSUS TIME OF WATER IMMERSION
FOR MATERIAL NO. 14 (COLORADO SHALE).

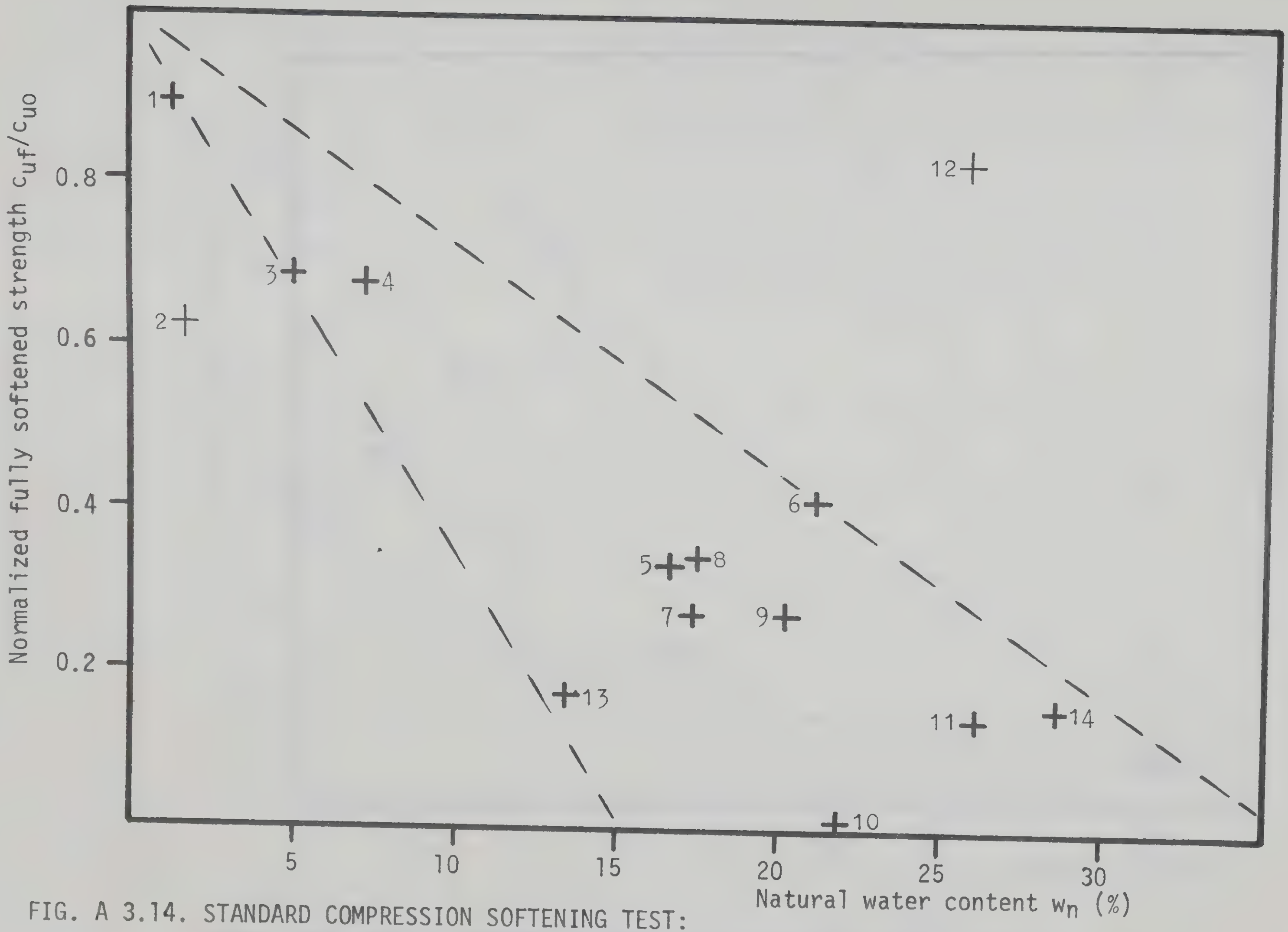


FIG. A 3.14. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED FULLY SOFTENED STRENGTH VERSUS NATURAL WATER CONTENT.

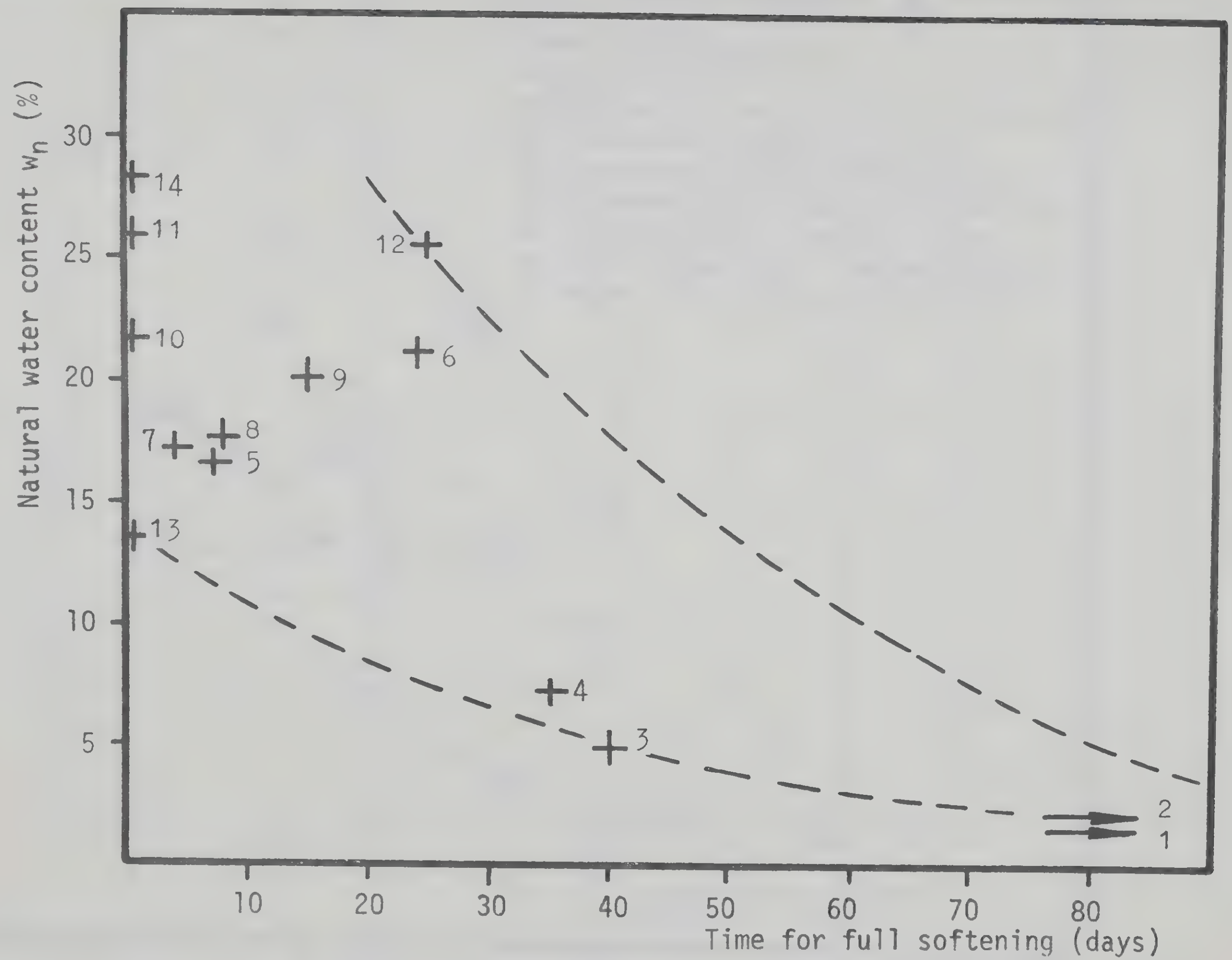


FIG. A 3.15. STANDARD COMPRESSION SOFTENING TEST:
INITIAL WATER CONTENT VERSUS TIME FOR FULL SOFTENING.

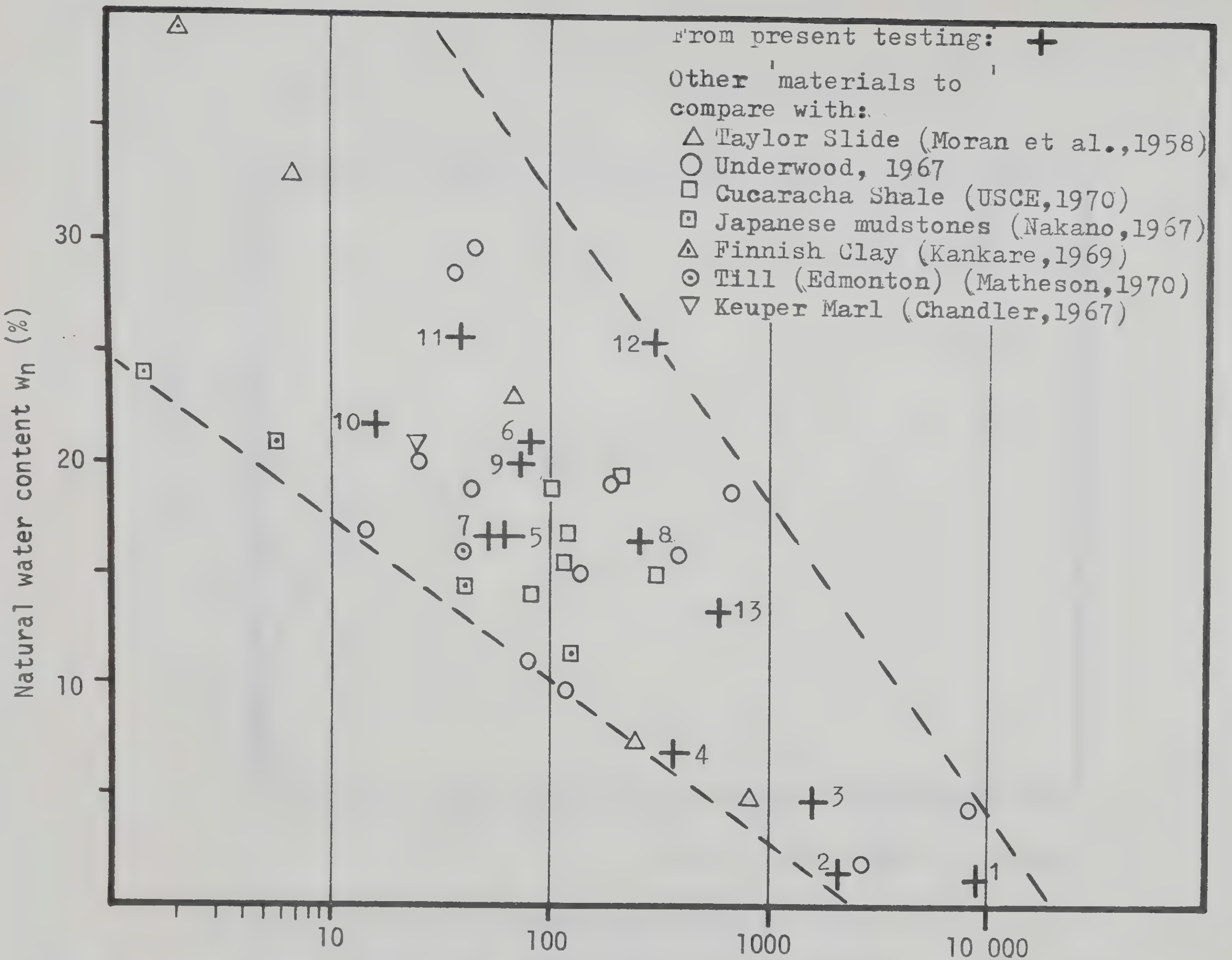


FIG. A 3.16. STANDARD COMPRESSION SOFTENING TEST: NATURAL WATER CONTENT VERSUS INITIAL UNDRAINED SHEAR STRENGTH.

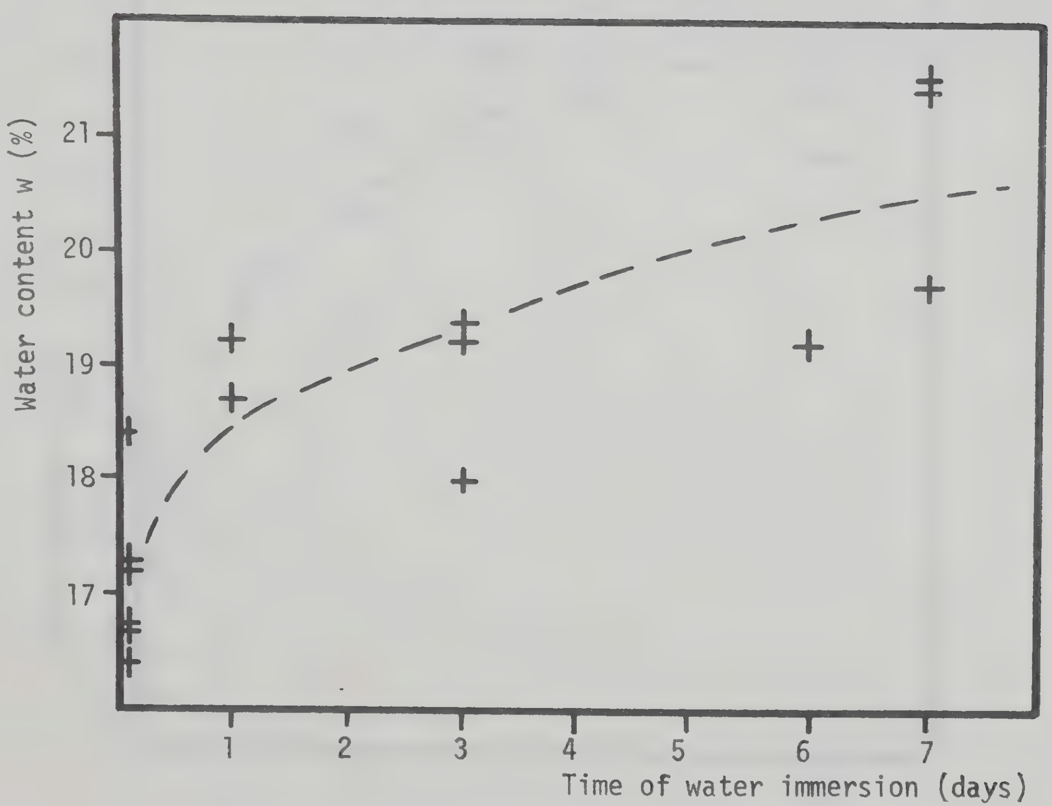


FIG. A 3.17. STANDARD COMPRESSION SOFTENING TEST:

CHANGE OF WATER CONTENT WITH TIME OF IMMERSION IN WATER

FOR MATERIAL NO. 5.

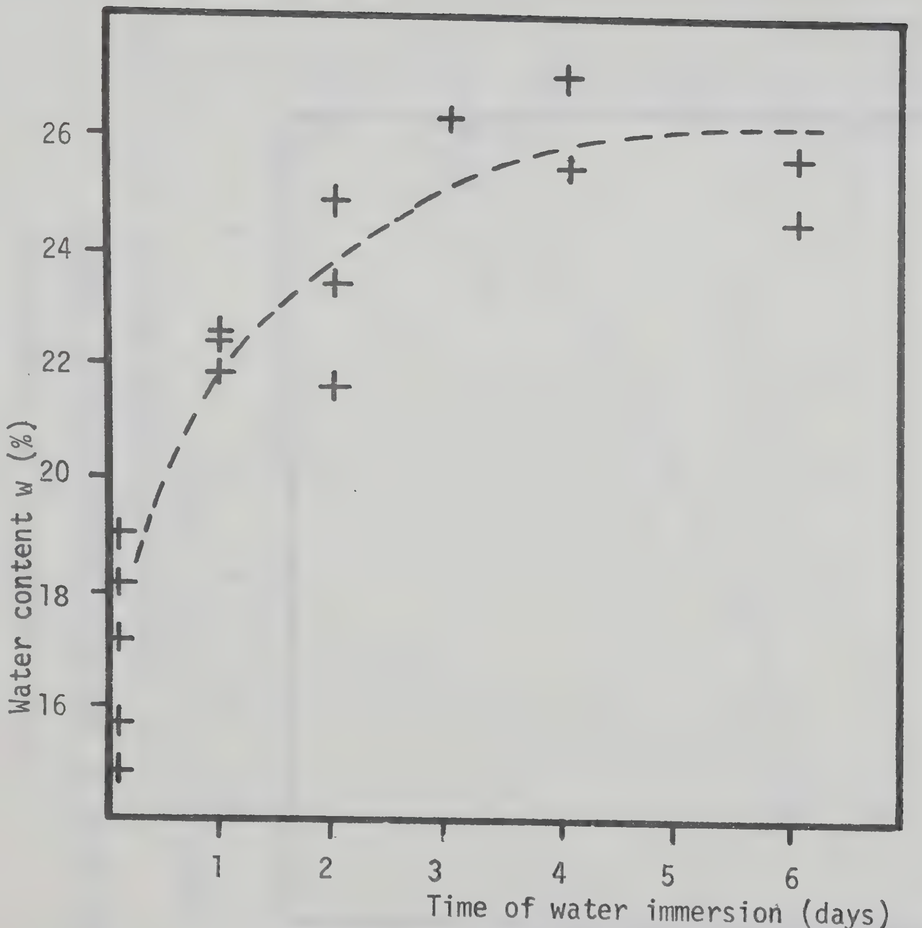


FIG. A 3.18. STANDARD COMPRESSION SOFTENING TEST:

CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION FOR MATERIAL NO. 7.

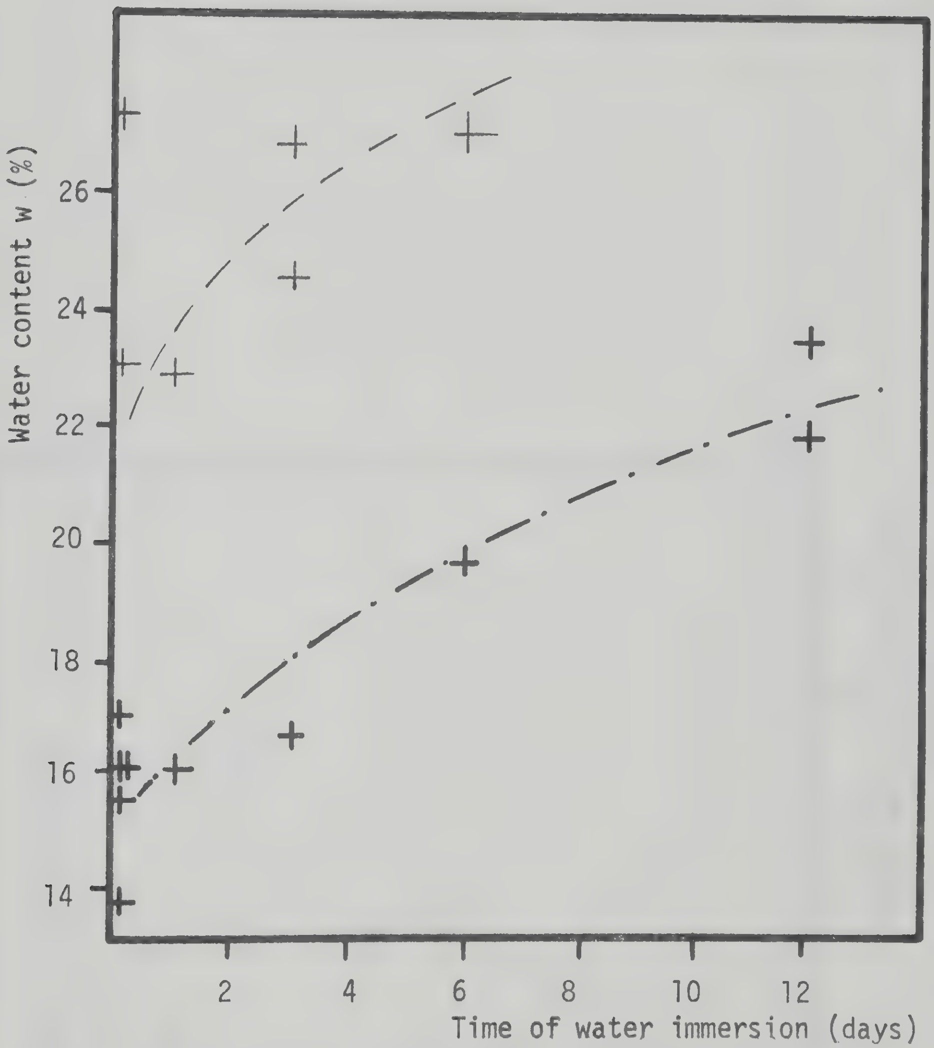


FIG. A 3.19. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION
FOR MATERIAL NO. 8 (BEARPAW SHALE).

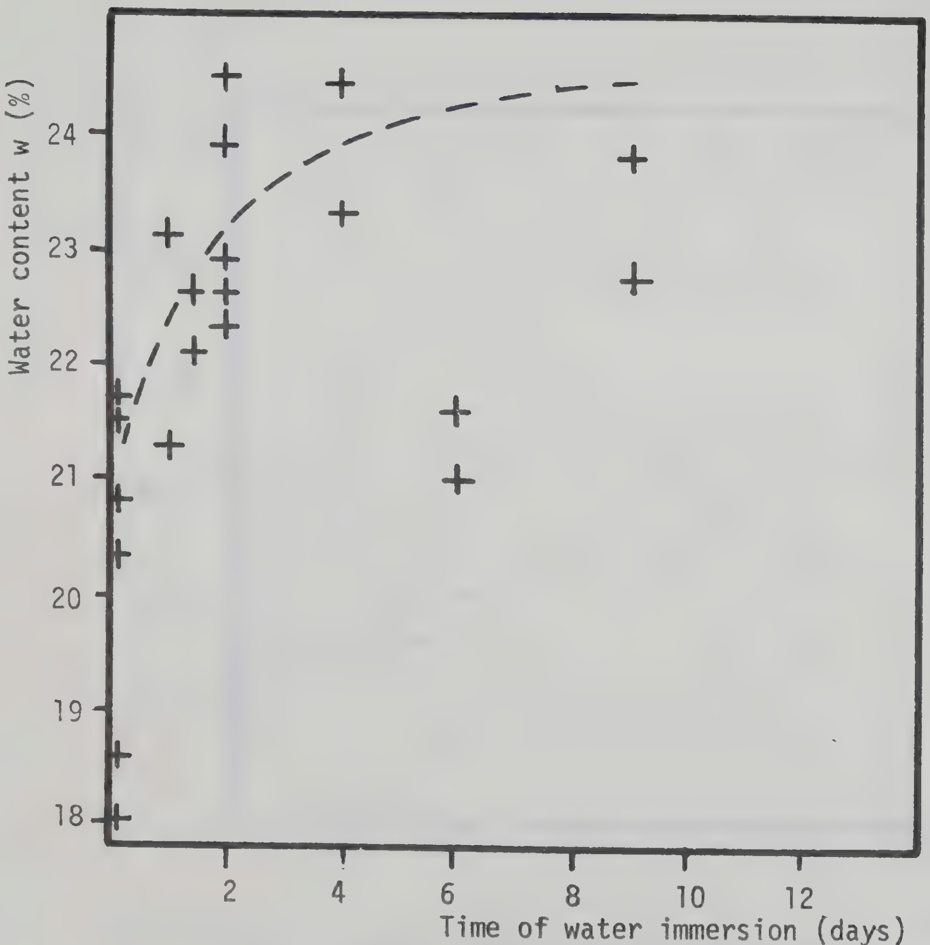


FIG. A 3.20. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION FOR MATERIAL NO. 9 (OXFORD CLAY).

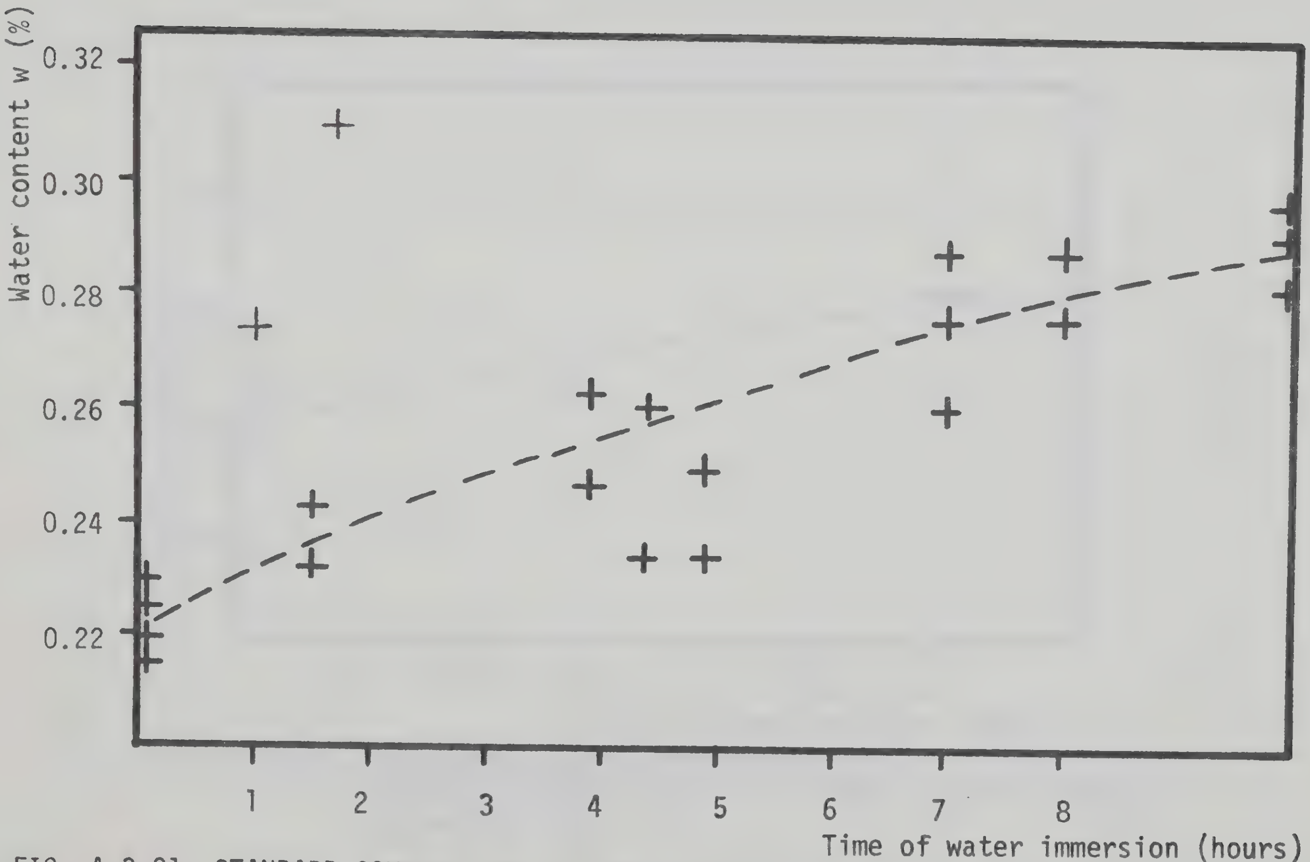


FIG. A 3.21. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION
FOR MATERIAL NO. 10 (GAULT CLAY).

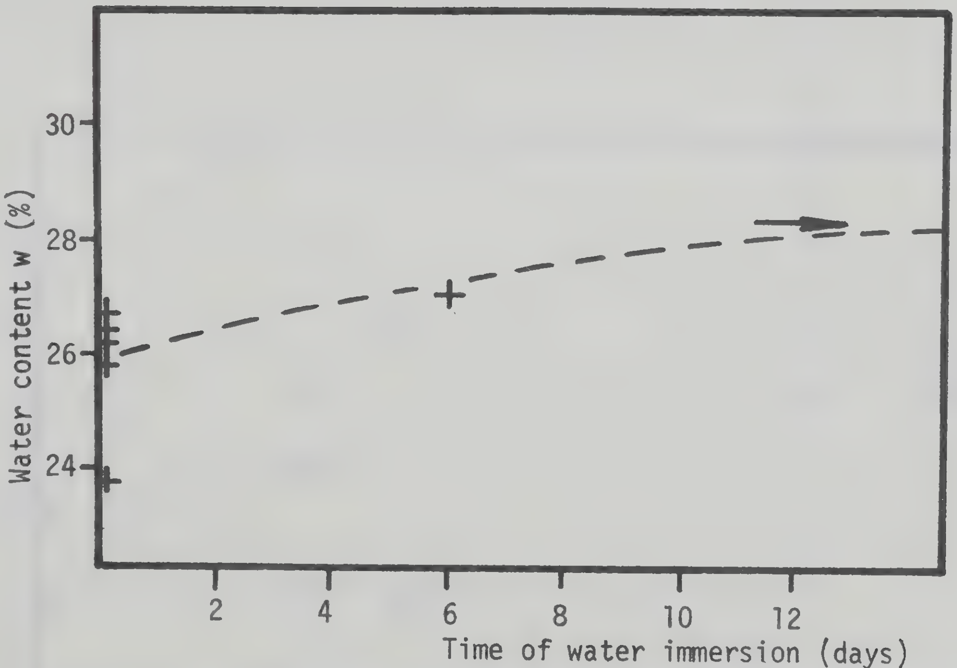


FIG. A 3.22. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION
FOR MATERIAL NO. 12.

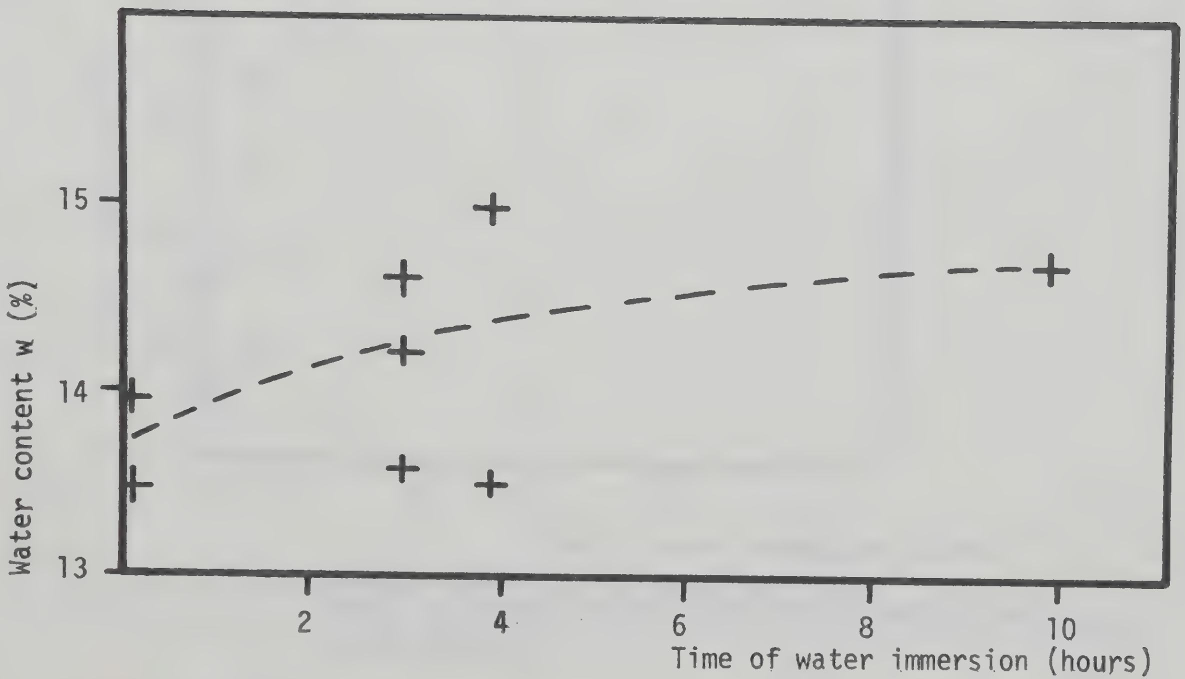


FIG. A 3.23. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION
FOR MATERIAL NO. 13.

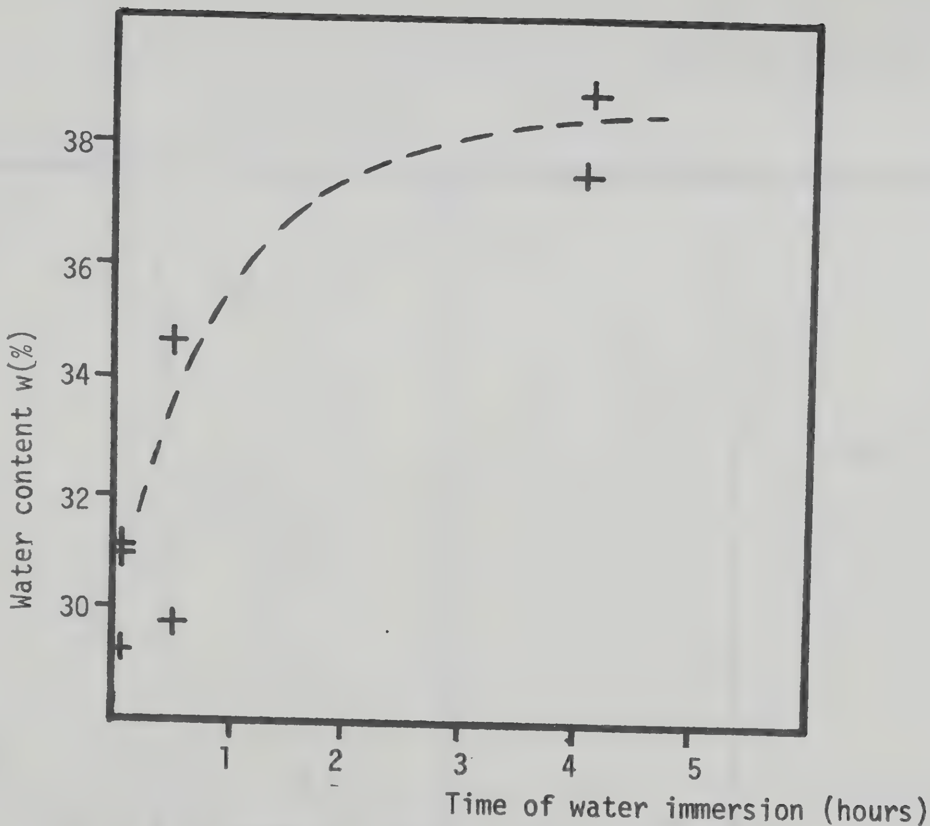


FIG. A 3.24. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF WATER CONTENT WITH TIME OF WATER IMMERSION
FOR MATERIAL NO. 14.

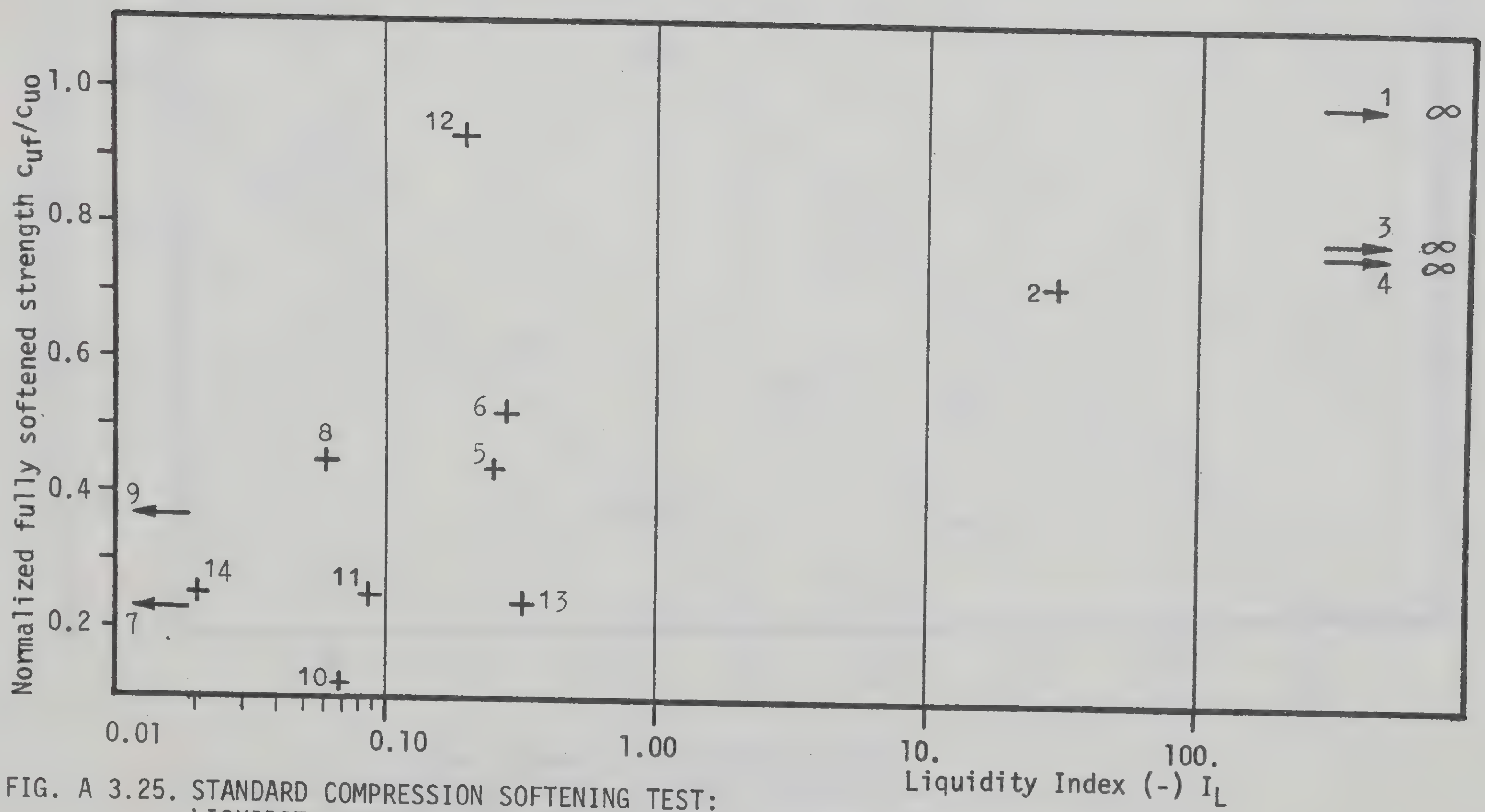


FIG. A 3.25. STANDARD COMPRESSION SOFTENING TEST:
LIQUIDITY INDEX VERSUS NORMALIZED FULLY SOFTENED STRENGTH.

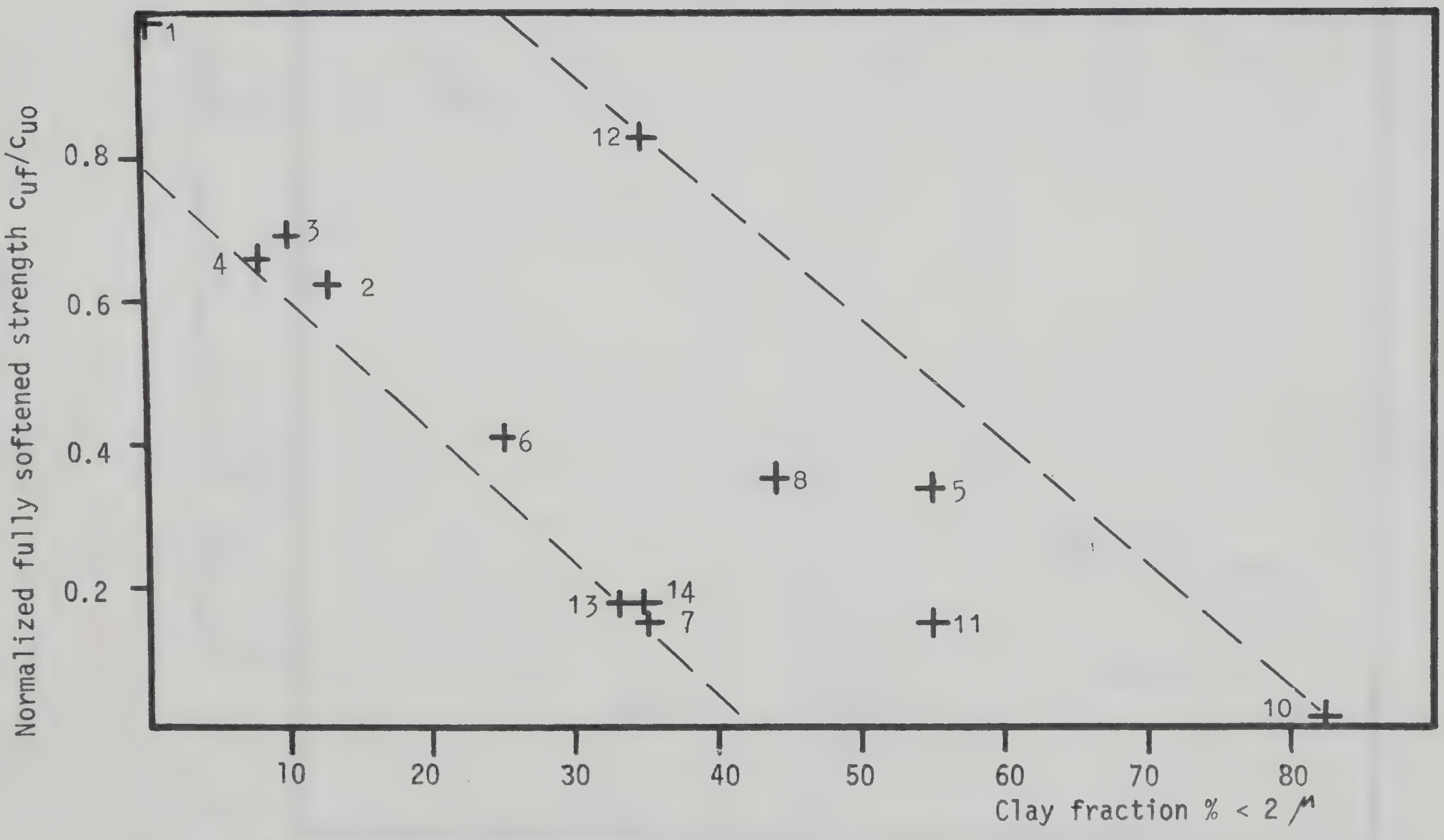


FIG. A 3.26. STANDARD COMPRESSION SOFTENING TEST:
NORMALIZED FULLY SOFTENED STRENGTH VERSUS CLAY FRACTION.

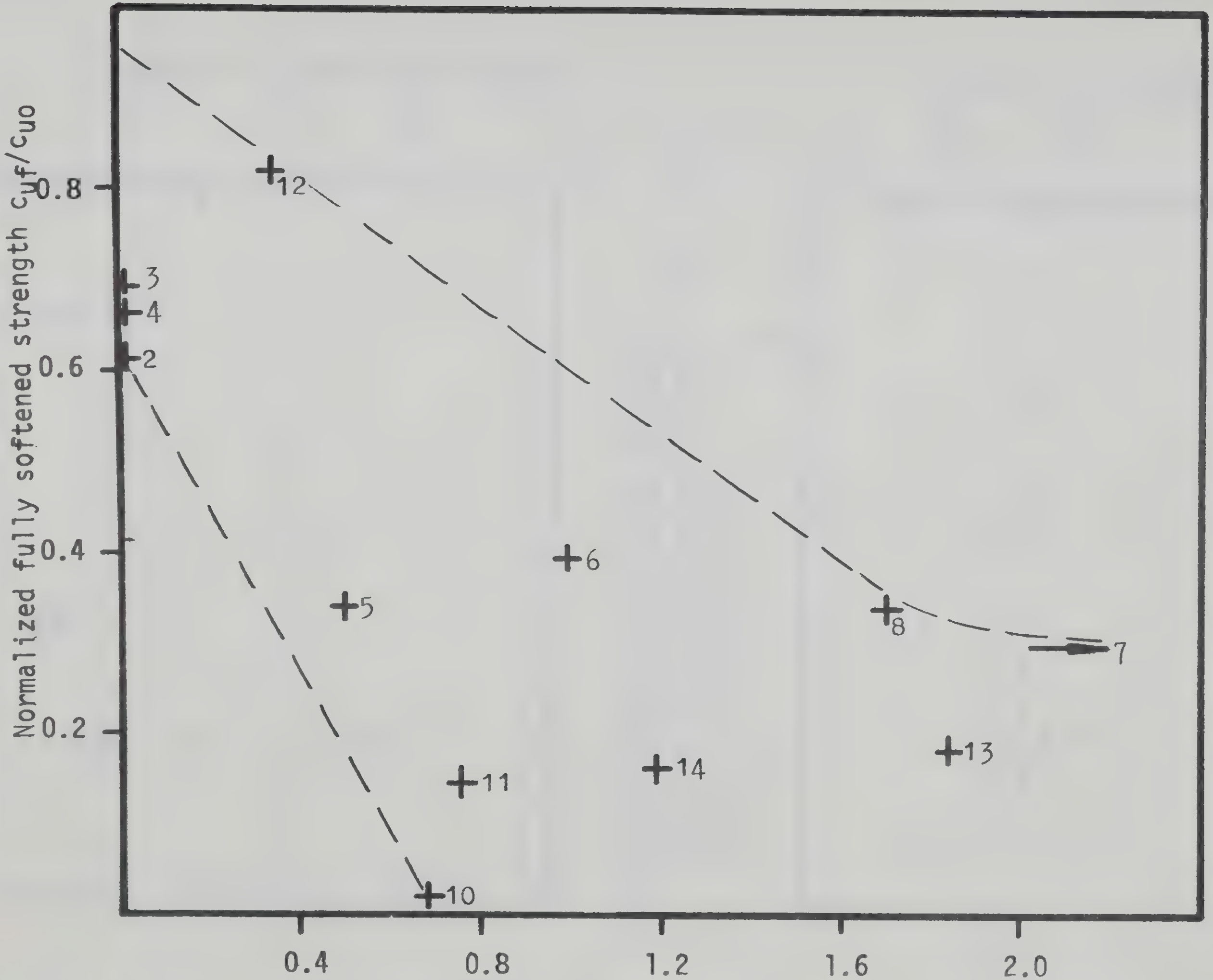


FIG. A 3.27. STANDARD COMPRESSION SOFTENING TEST: NORMALIZED FULLY SOFTENED STRENGTH VERSUS ACTIVITY.

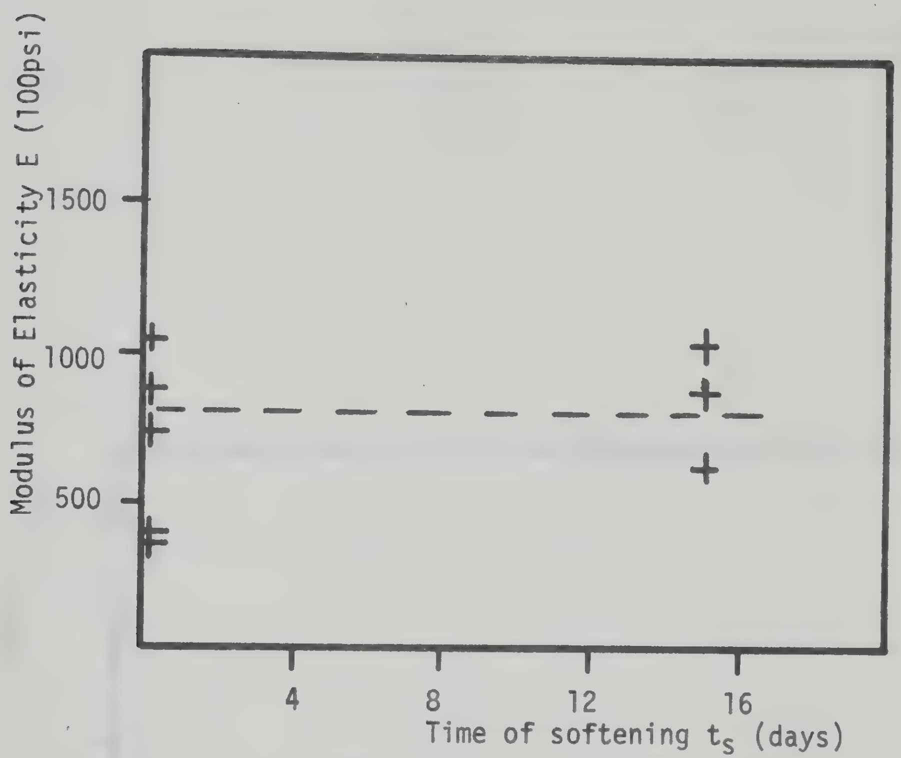


FIG. A 3.28. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 3.

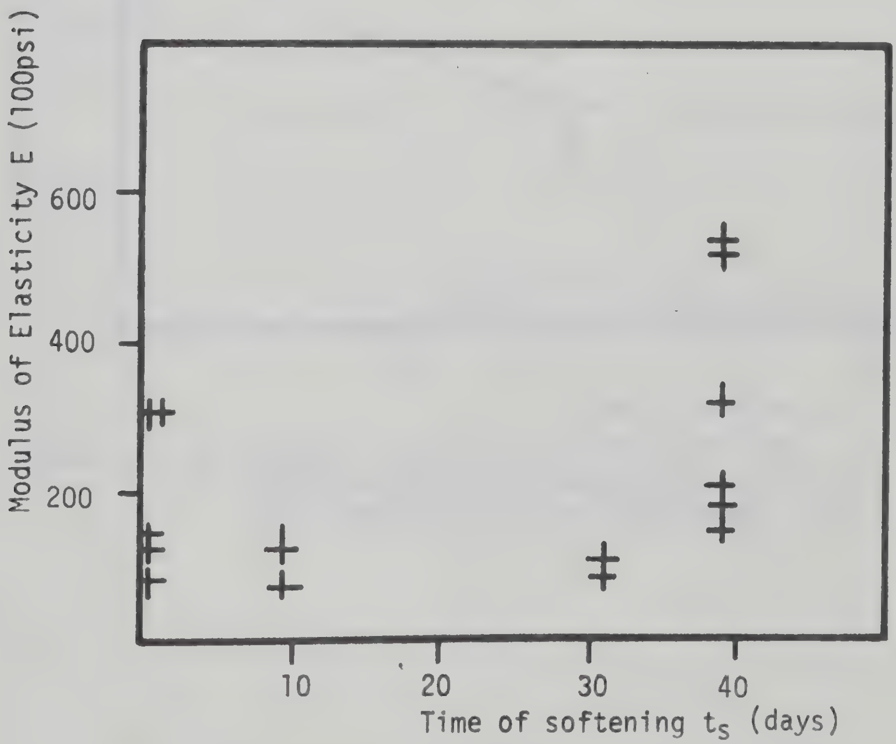


FIG. A 3.29. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 4.

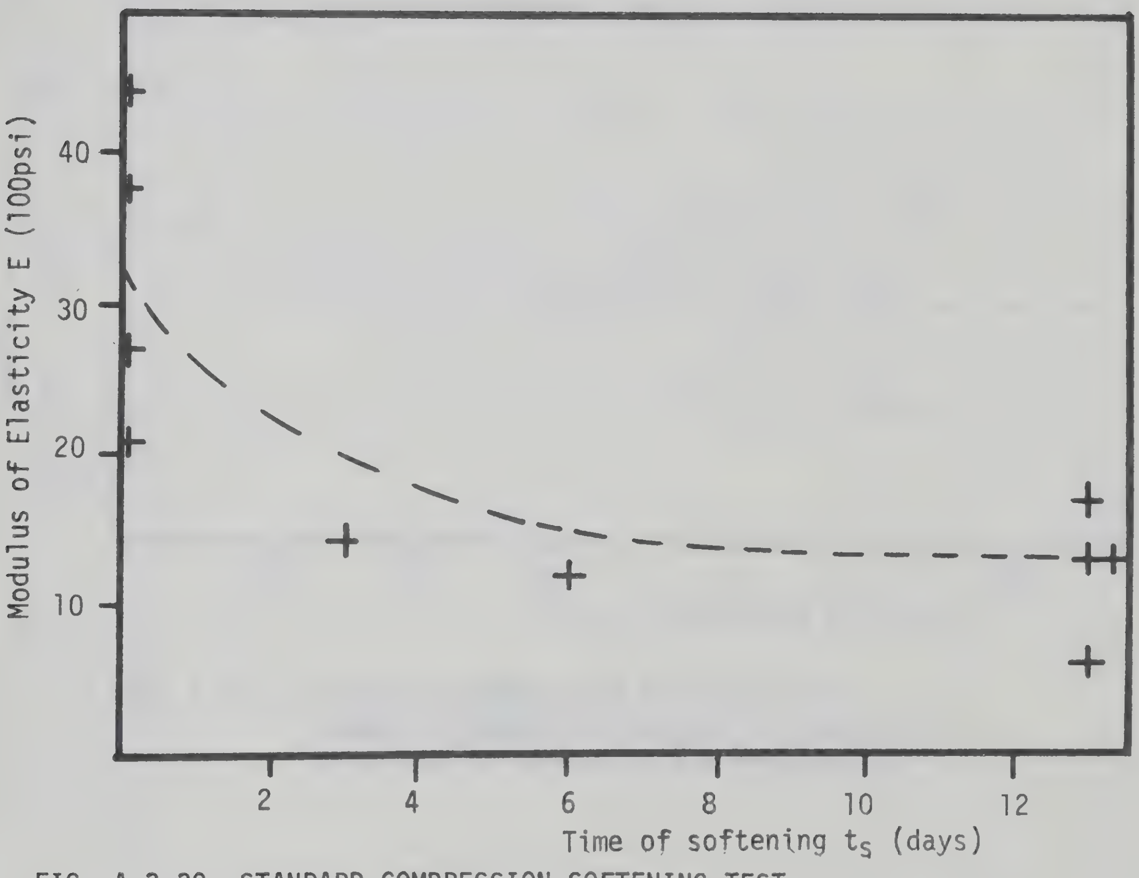


FIG. A 3.30. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF UNDRAINED MODULUS OF ELASTICITY
WITH TIME OF SOFTENING FOR MATERIAL NO. 5.

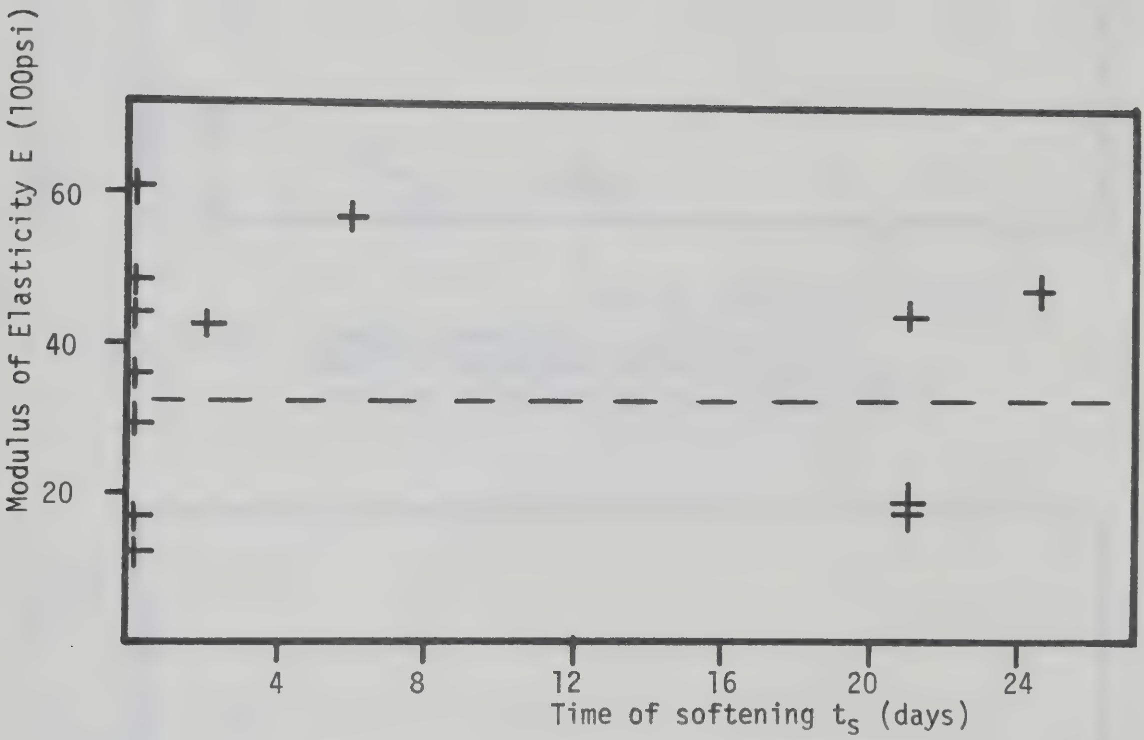


FIG. A 3.31. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 6.

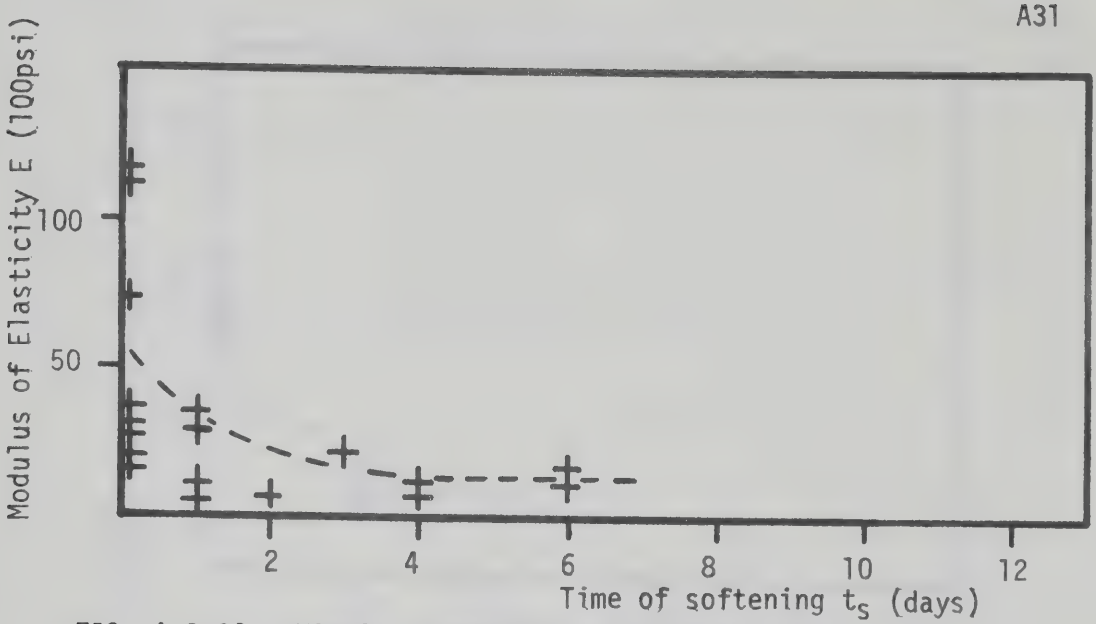


FIG. A 3.32. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF UNDRAINED MODULUS OF ELASTICITY
WITH TIME OF SOFTENING FOR MATERIAL NO. 7

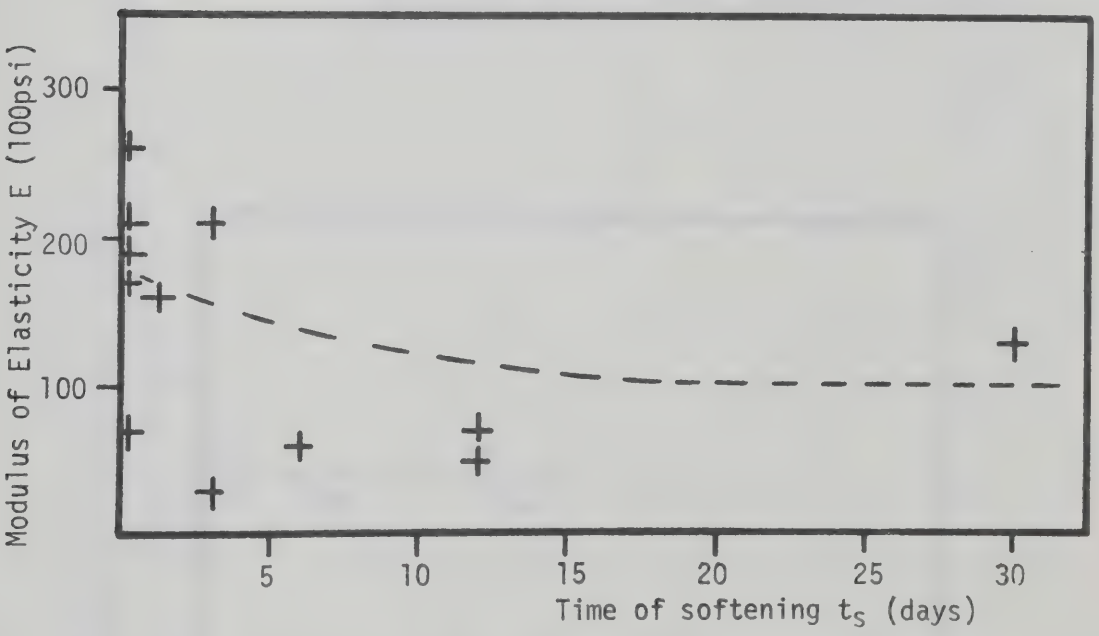
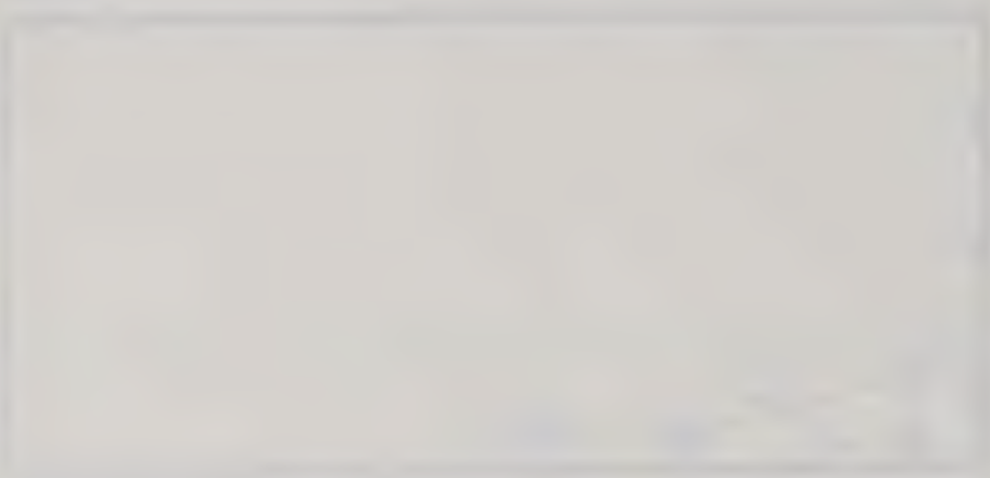
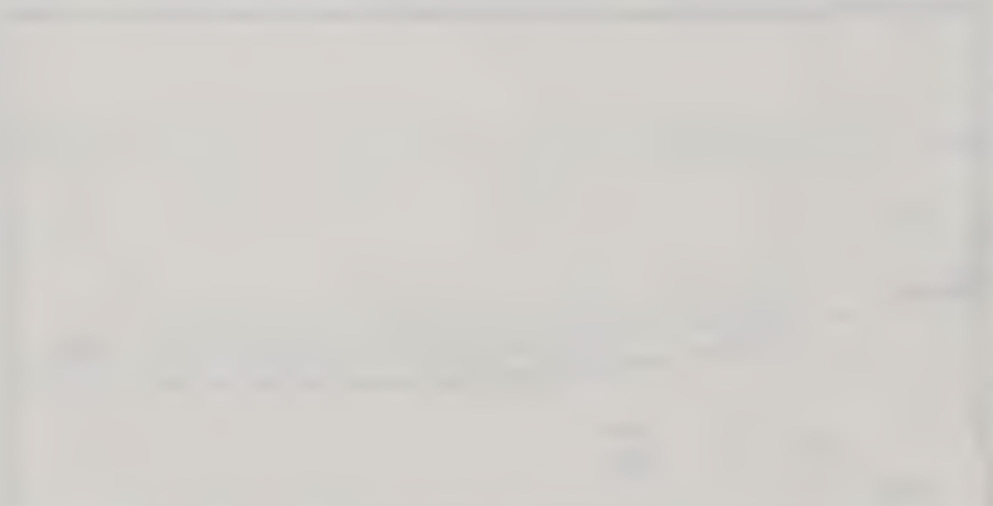


FIG. A 3.33. STANDARD COMPRESSION SOFTENING TEST:
CHANGE OF UNDRAINED MODULUS OF ELASTICITY
WITH TIME OF SOFTENING FOR MATERIAL NO. 8.





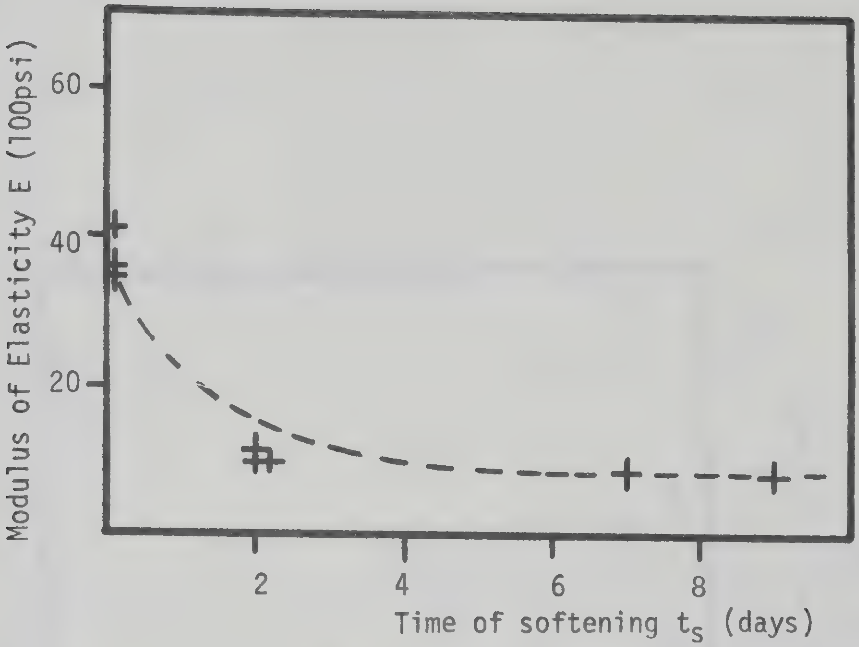


FIG. A 3.34. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 9 (OXFORD CLAY).

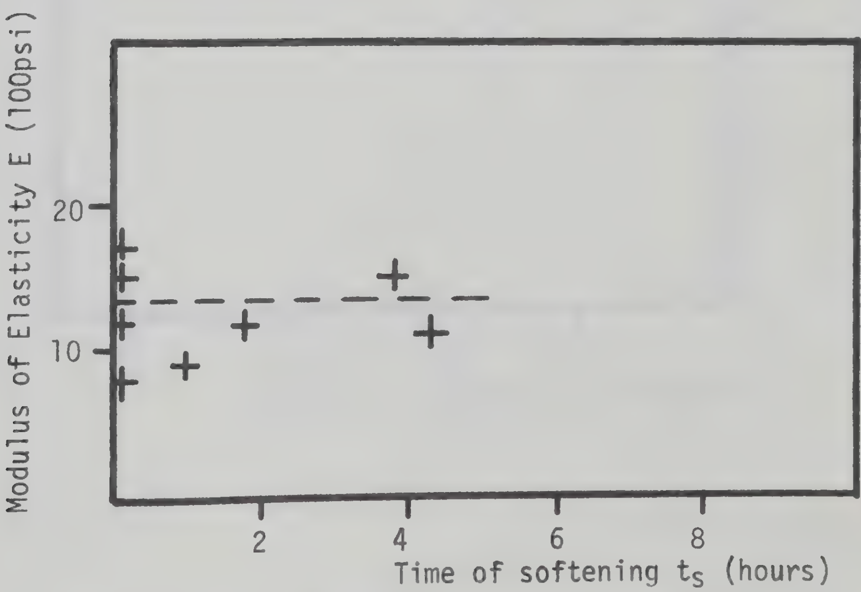


FIG. A 3.35. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 10 (GAULT CLAY).

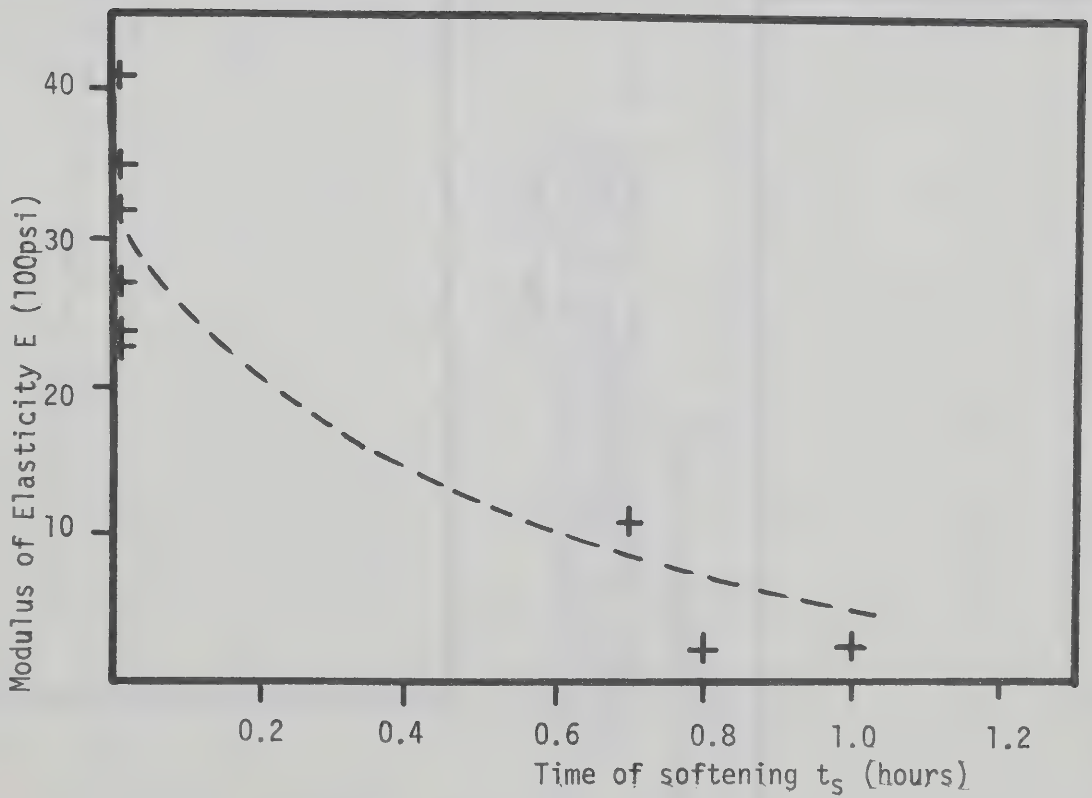


FIG. A 3.36. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 11 (LONDON CLAY).

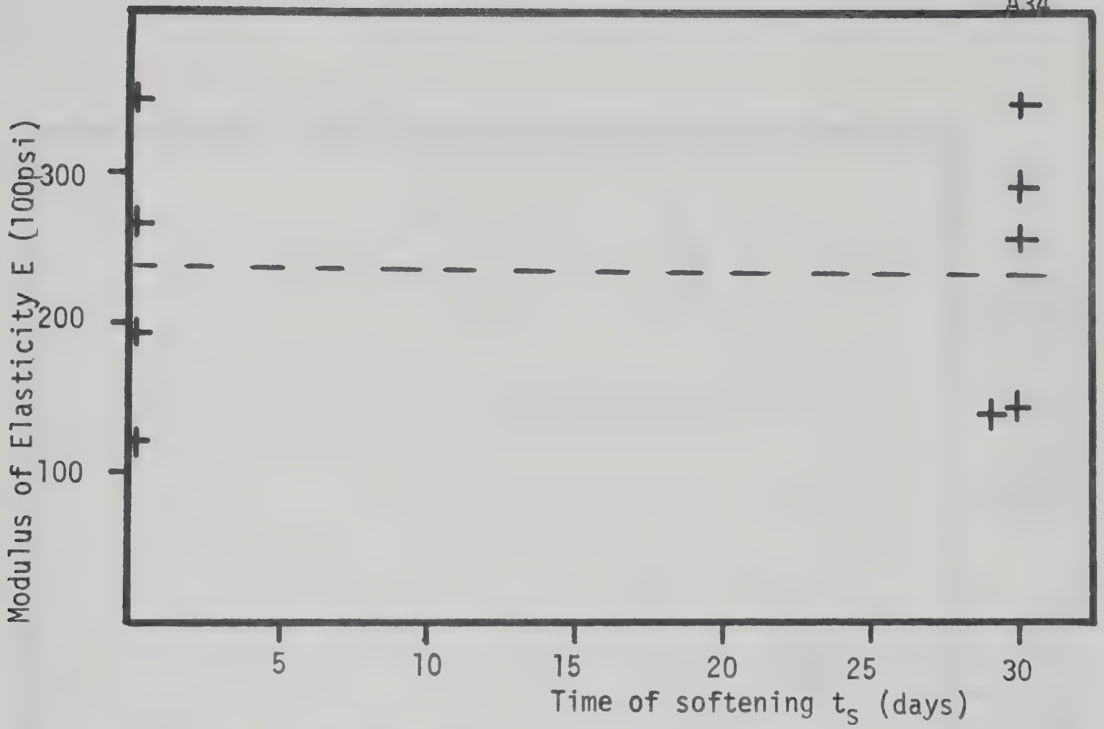


FIG. A 3.37. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 12 (PIERRE SHALE).

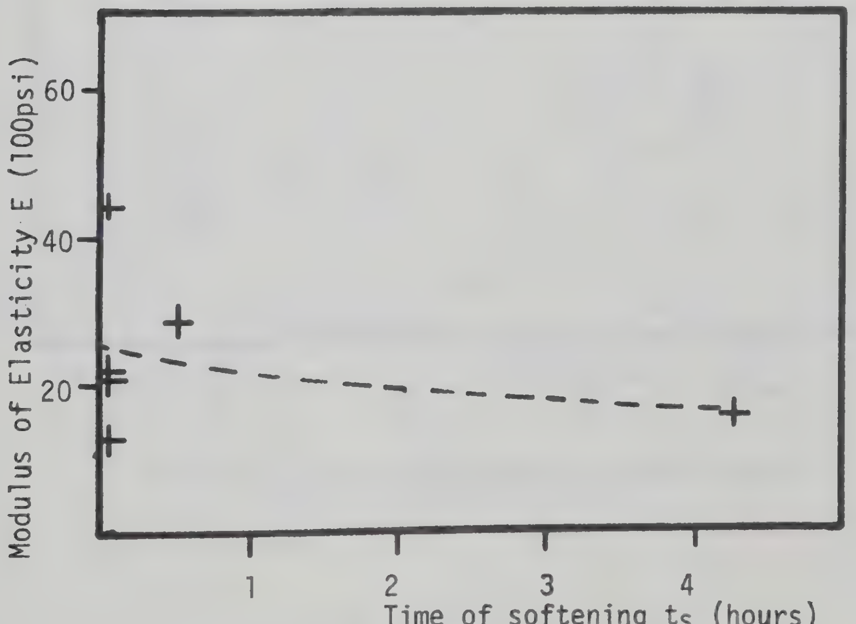


FIG. A 3.38. STANDARD COMPRESSION SOFTENING TEST: CHANGE OF UNDRAINED MODULUS OF ELASTICITY WITH TIME OF SOFTENING FOR MATERIAL NO. 14 (COLORADO SHALE).

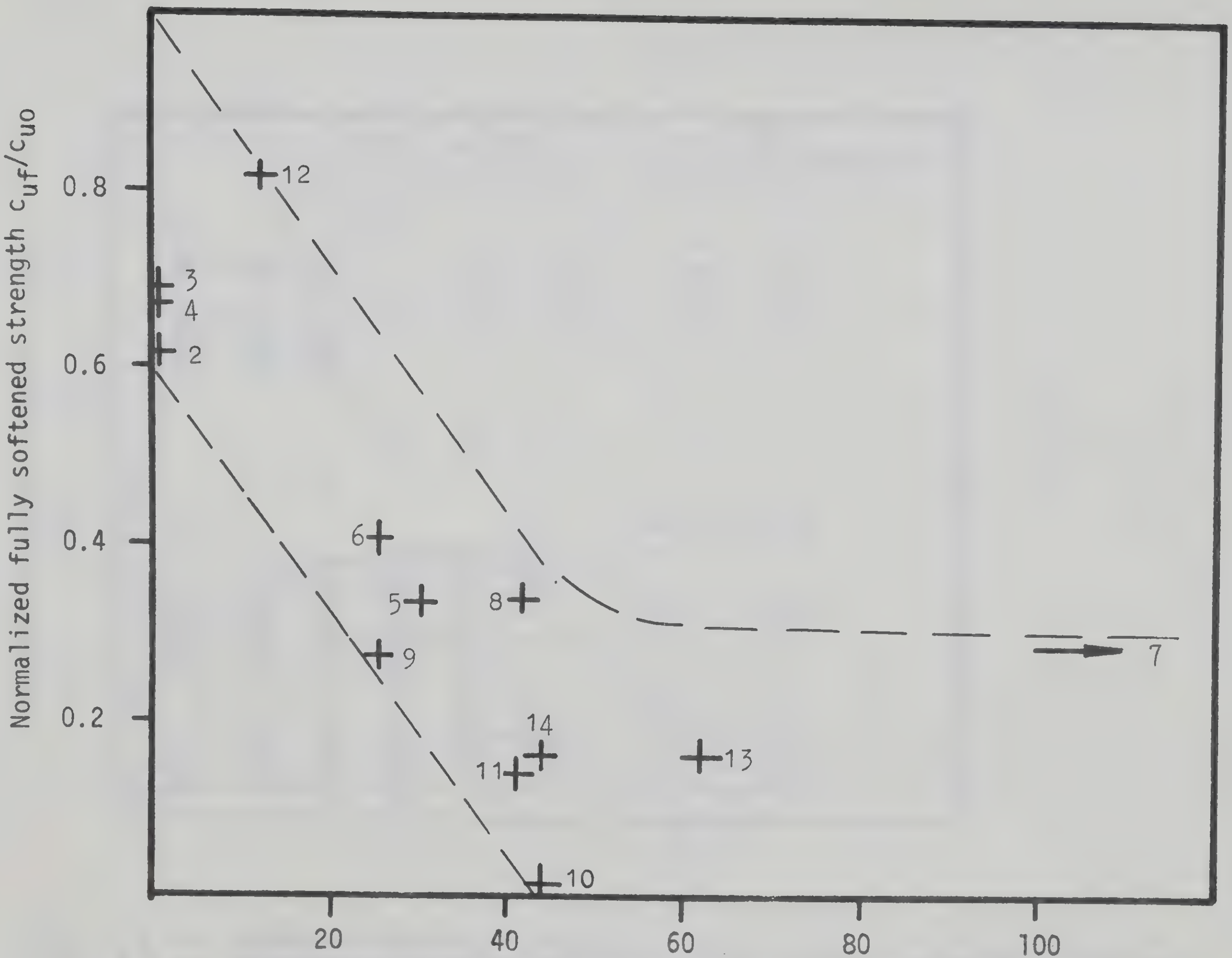


FIG. A 3.39. STANDARD COMPRESSION SOFTENING TEST: NORMALIZED FULLY SOFTENED STRENGTH VERSUS PLASTICITY INDEX.

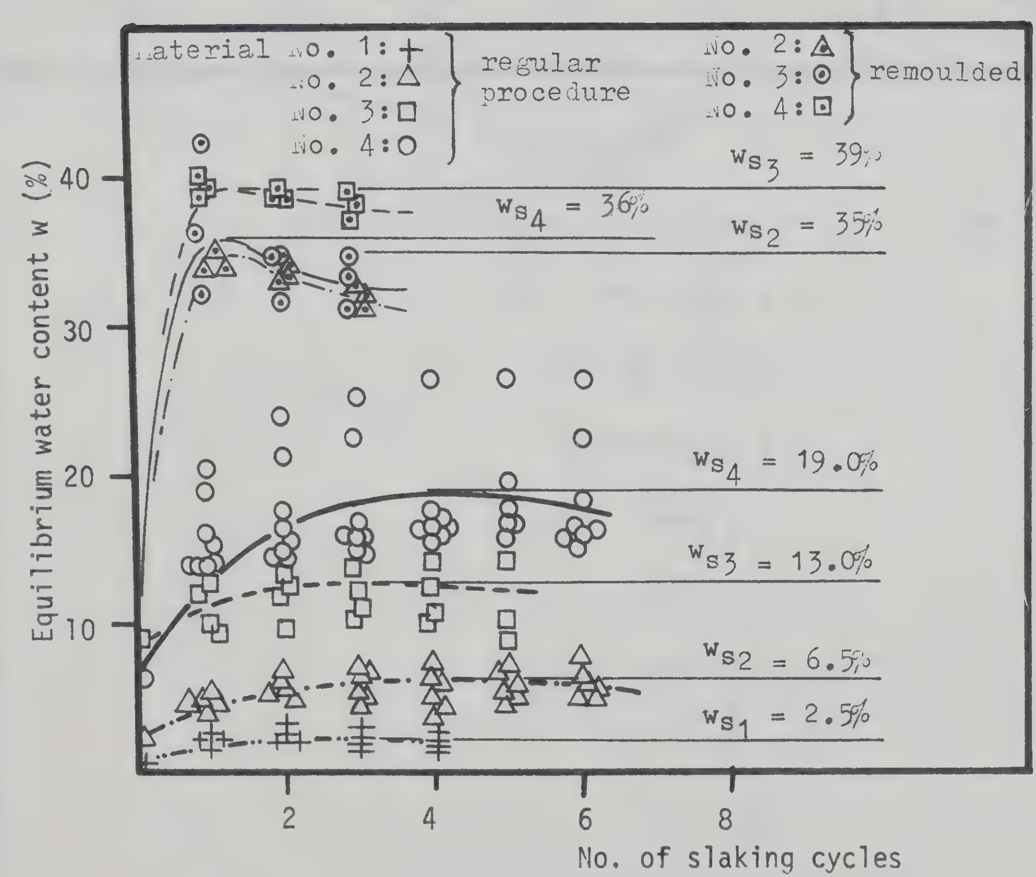


FIG. A 3.40. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE VERSUS NUMBER OF SLAKING CYCLES FOR MATERIALS NO. 1, 2, 3 AND 4.

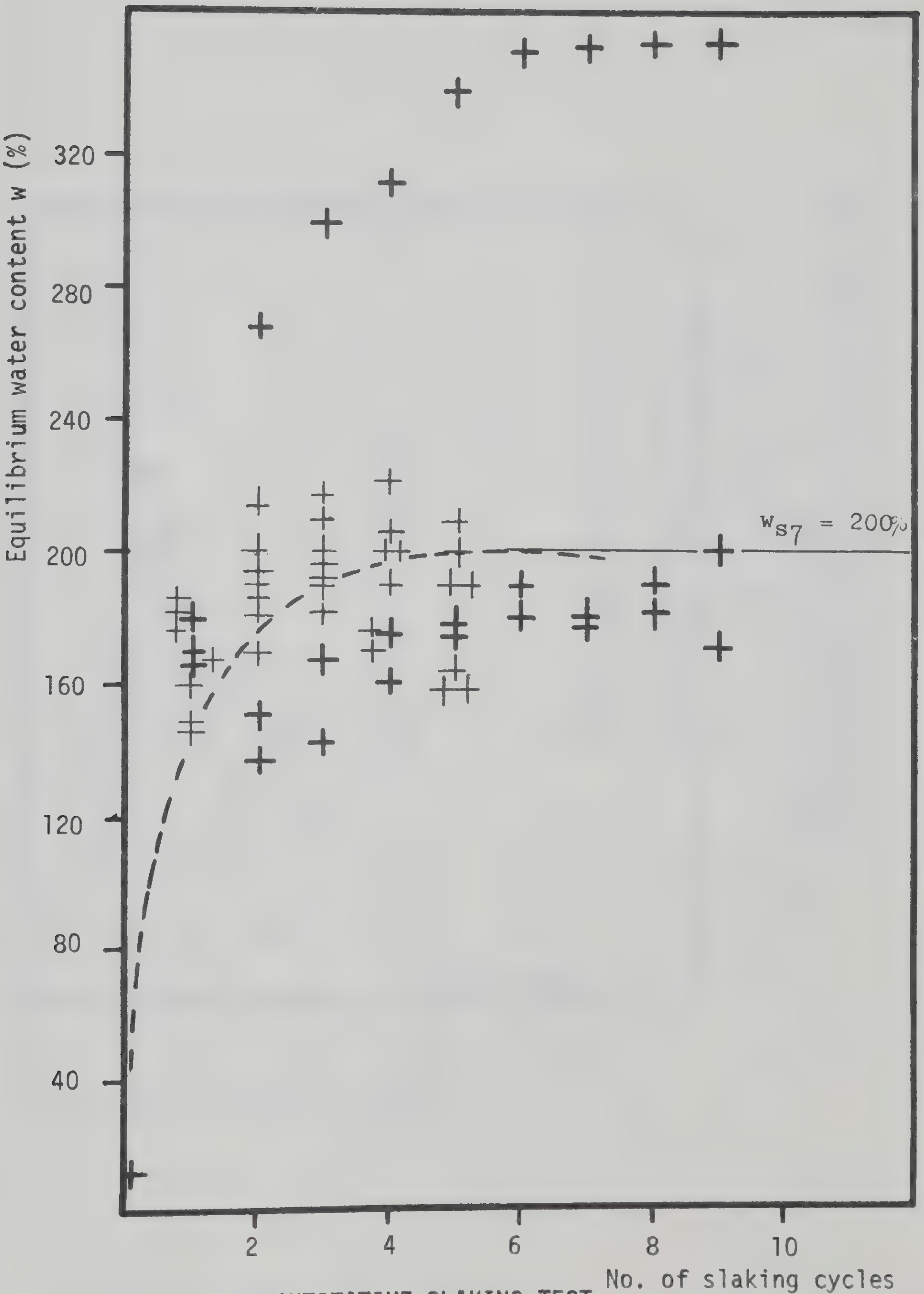


FIG. A 3.41. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 7.

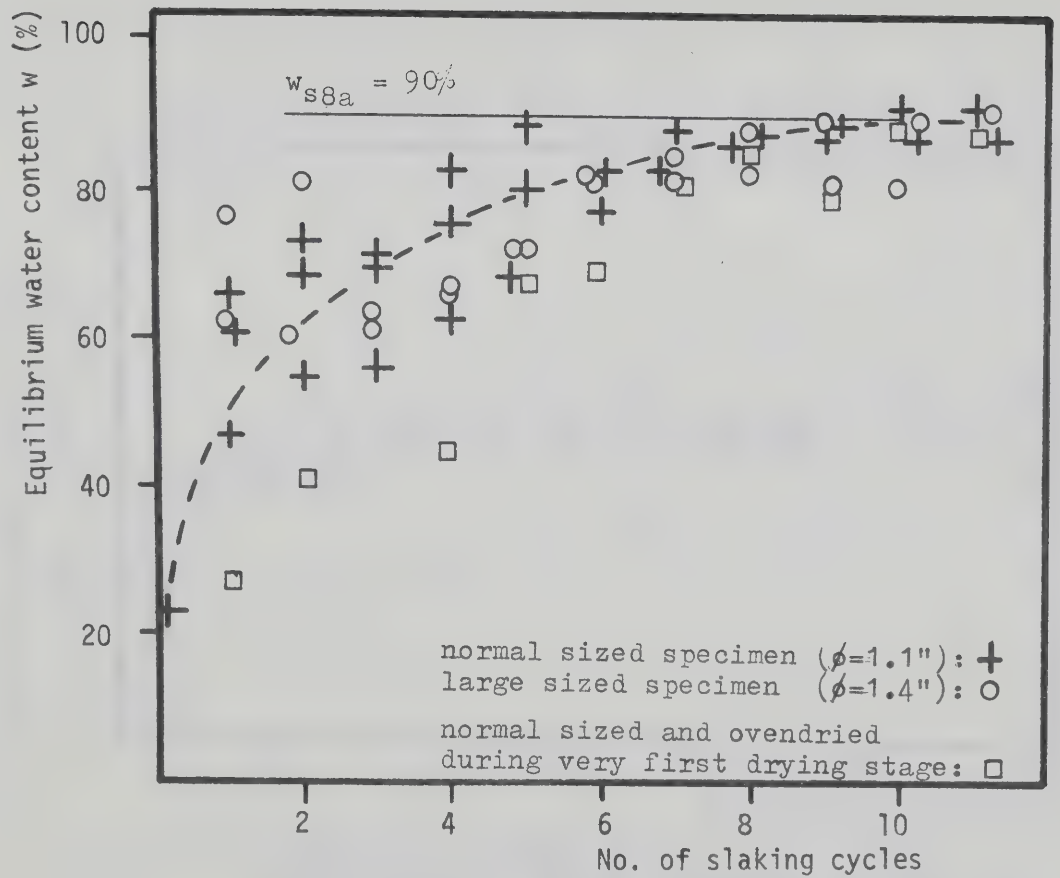


FIG. A 3.42. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 8a.

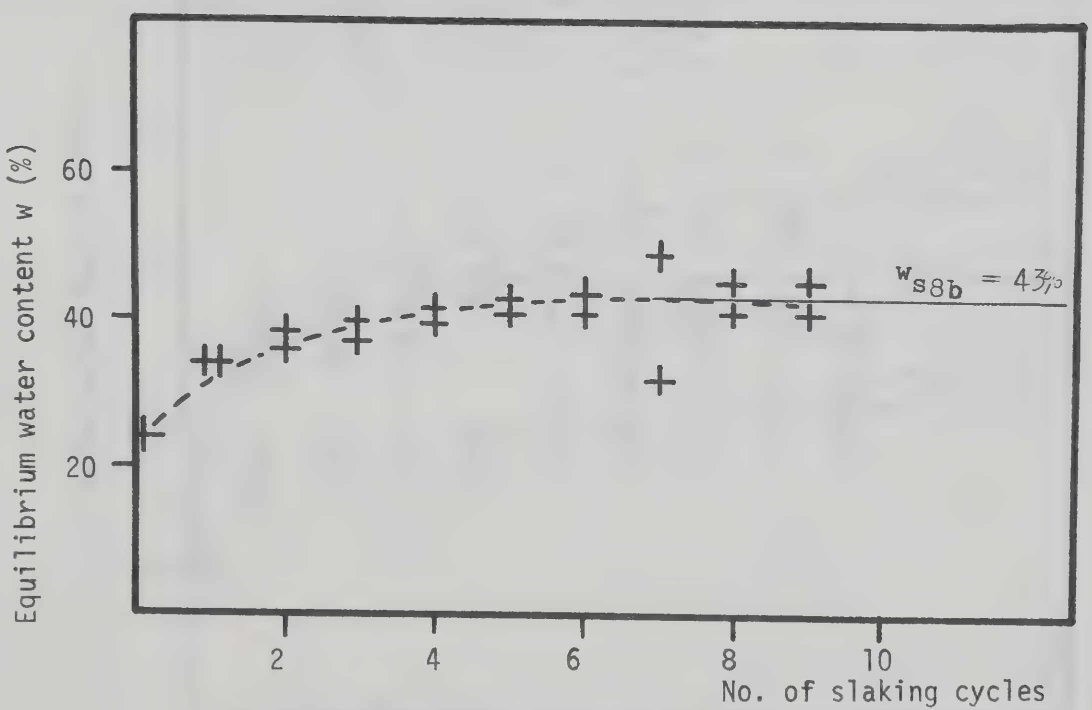


FIG. A 3.43. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 8b.

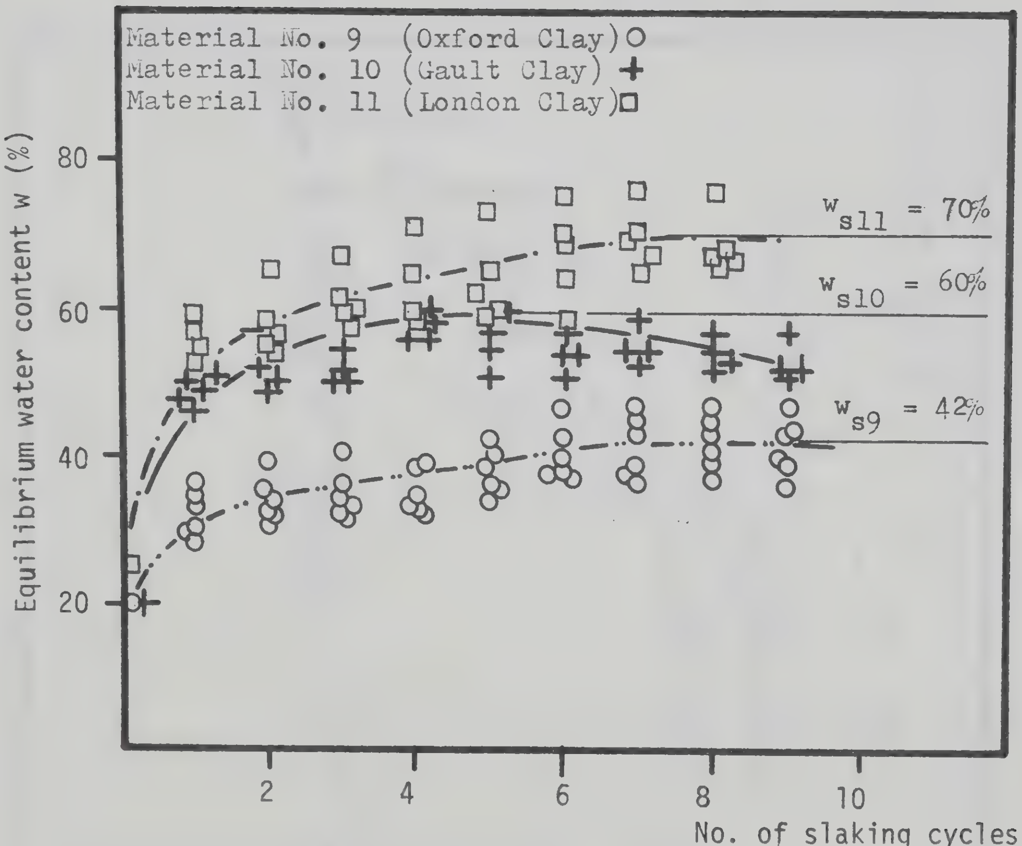


FIG. A 3.44. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIALS NO. 9, 10 AND 11.

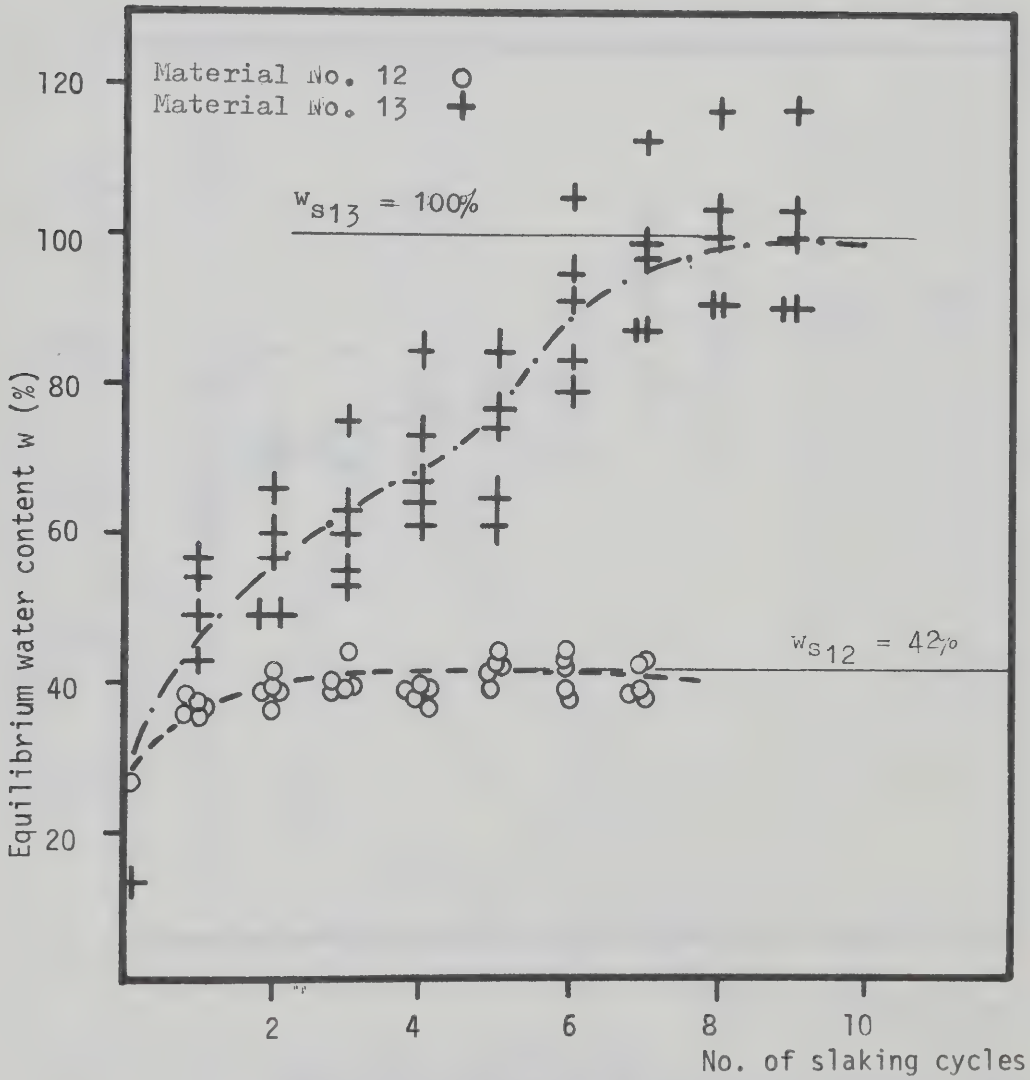


FIG. A 3.45. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE VERSUS NUMBER OF SLAKING CYCLES FOR MATERIALS 12 AND 13.

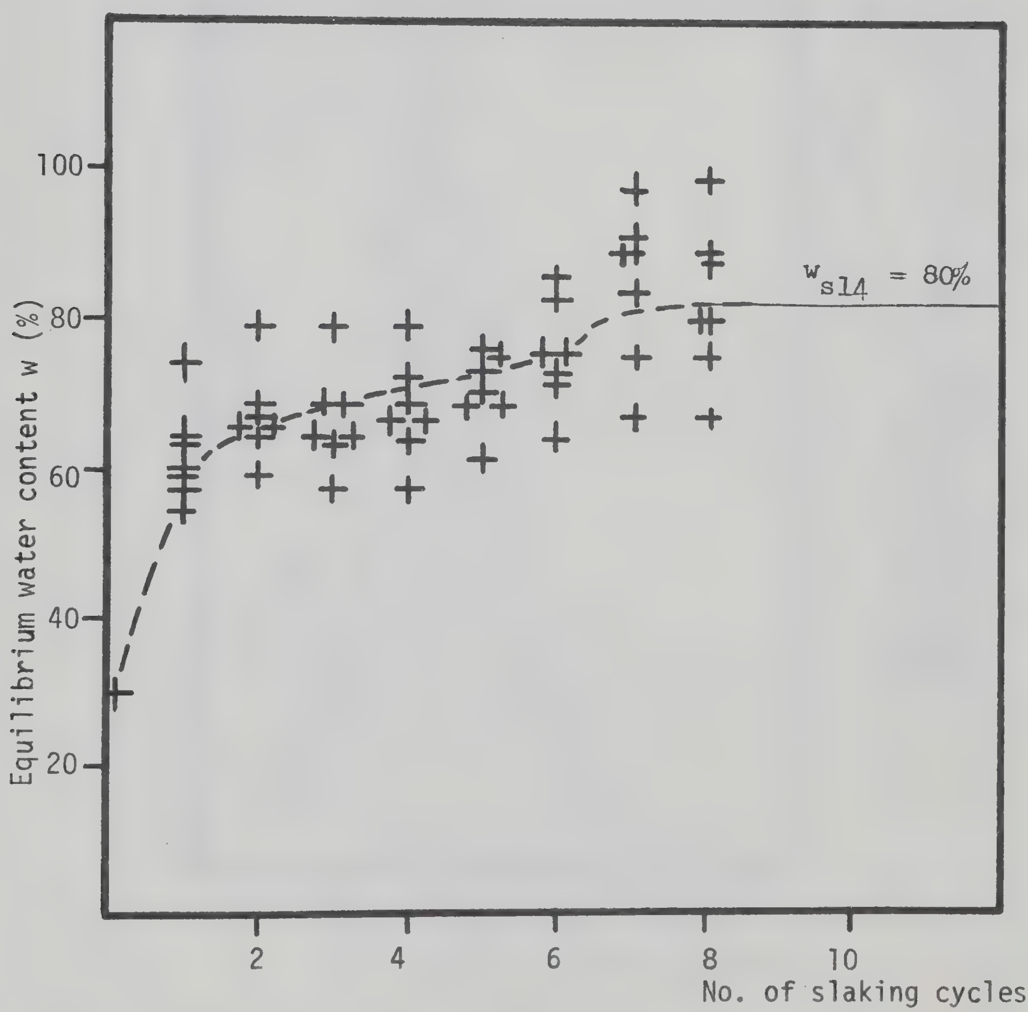


FIG. A 3.46. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 14.

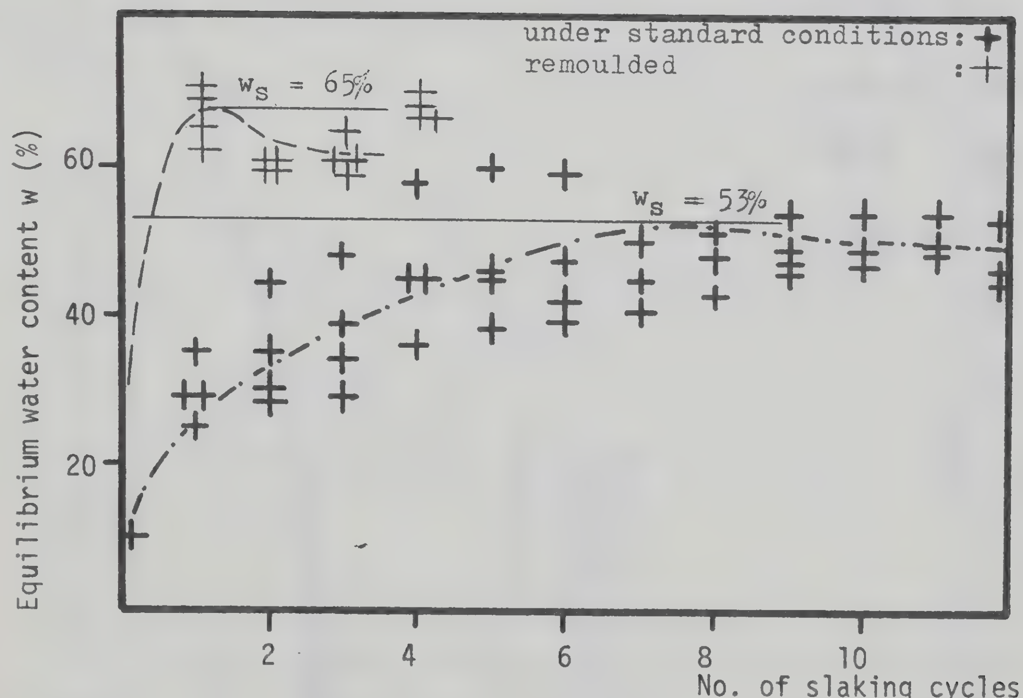


FIG. A 3.47. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 15.

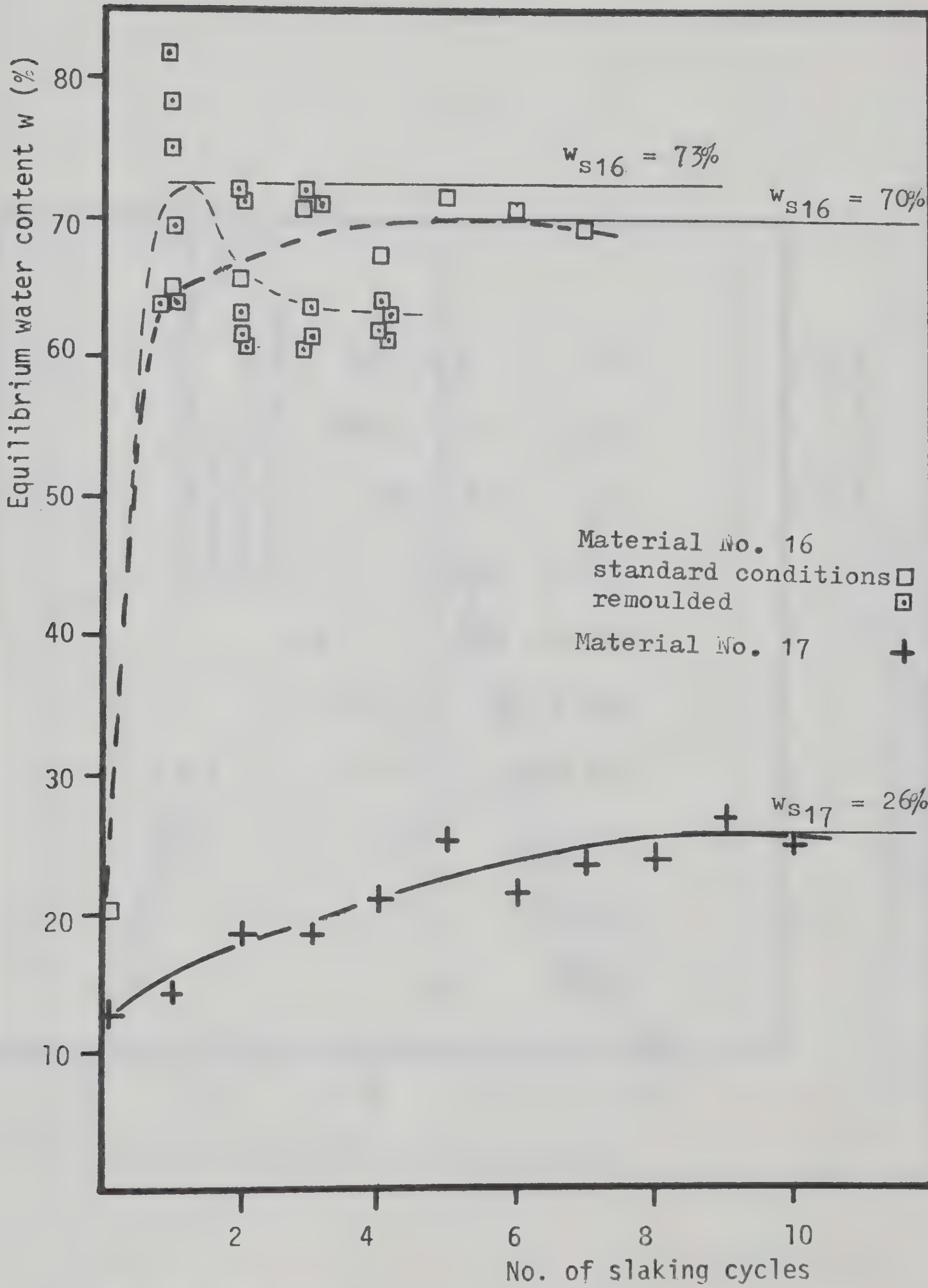


FIG. A 3.48. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE VERSUS NUMBER OF SLAKING CYCLES FOR MATERIALS NO. 16 AND 17.

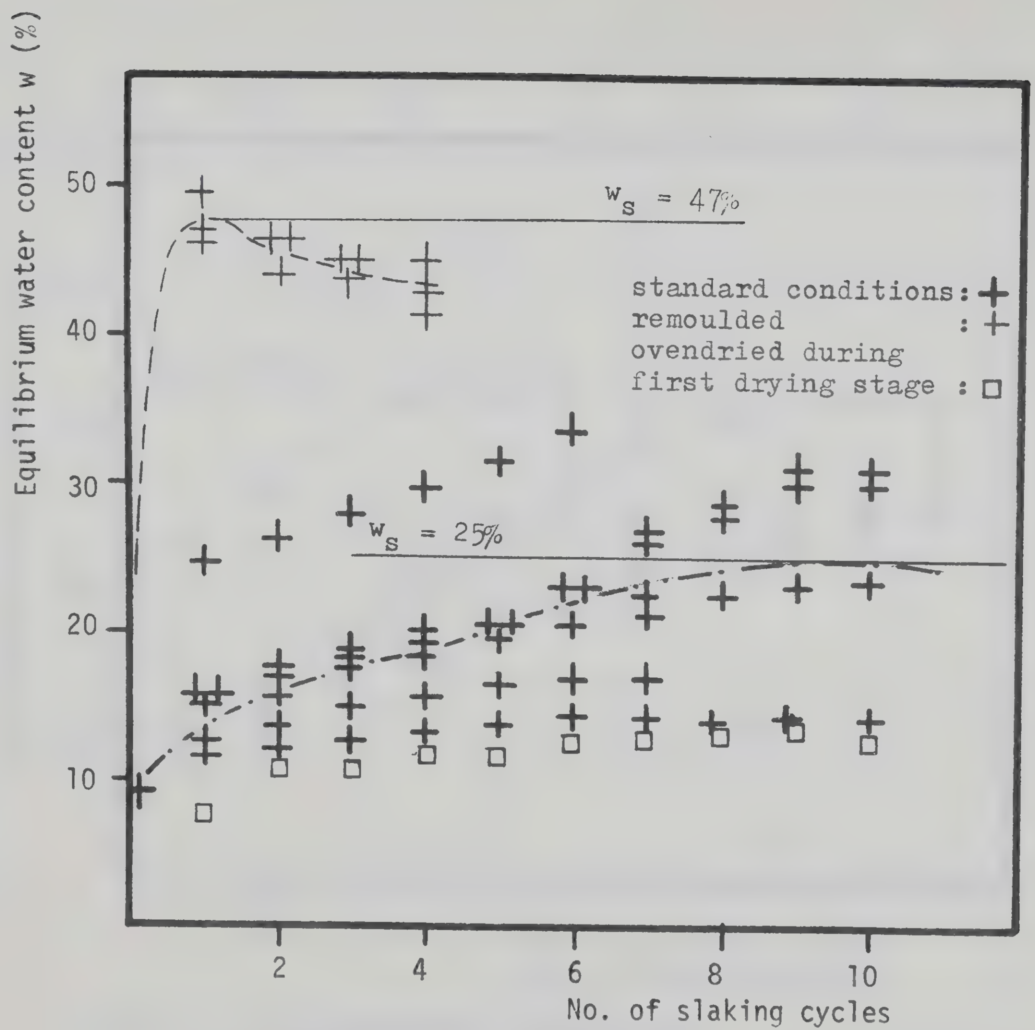


FIG. A 3.49. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 18.

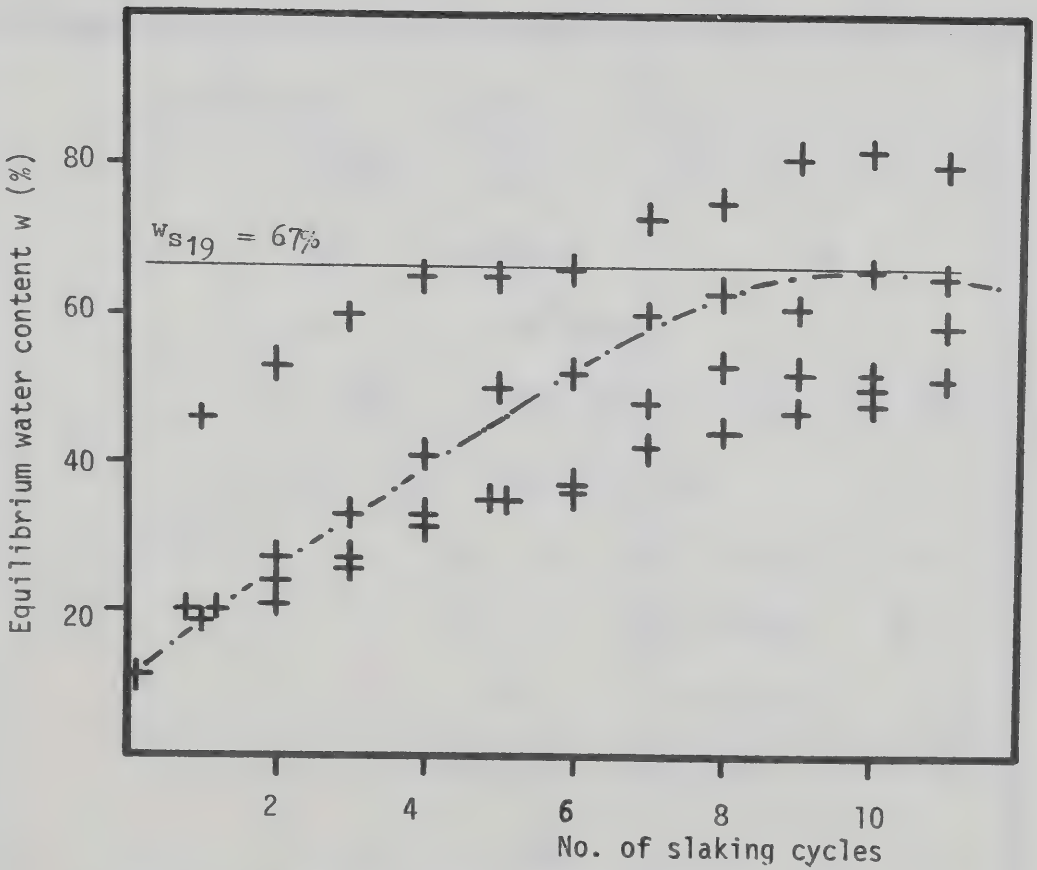


FIG. A 3.50. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 19.

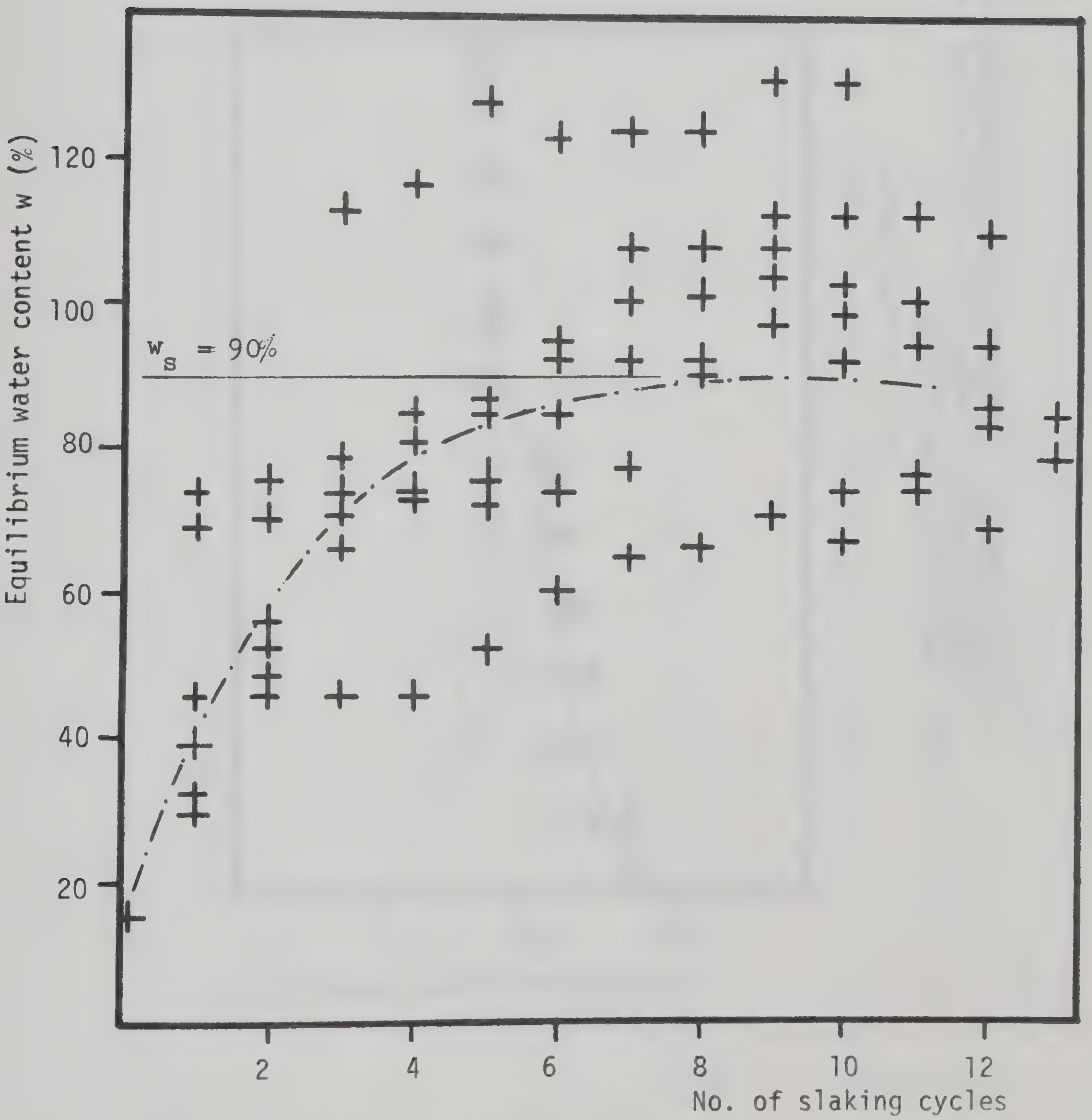


FIG. A 3.51. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 20.

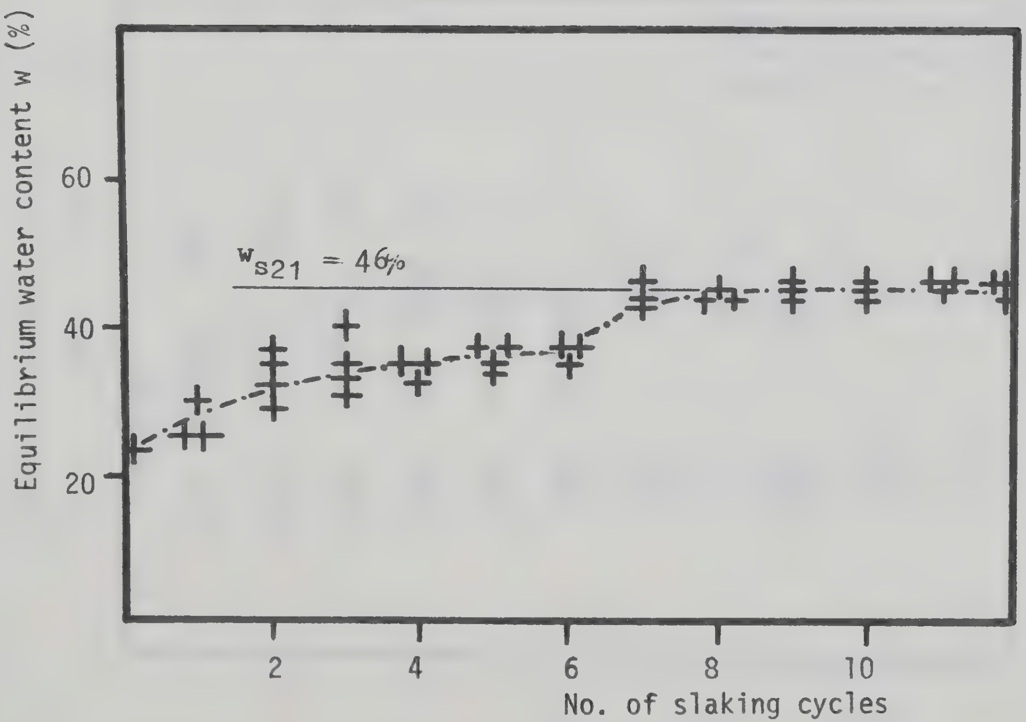


FIG. A 3.52. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 21.

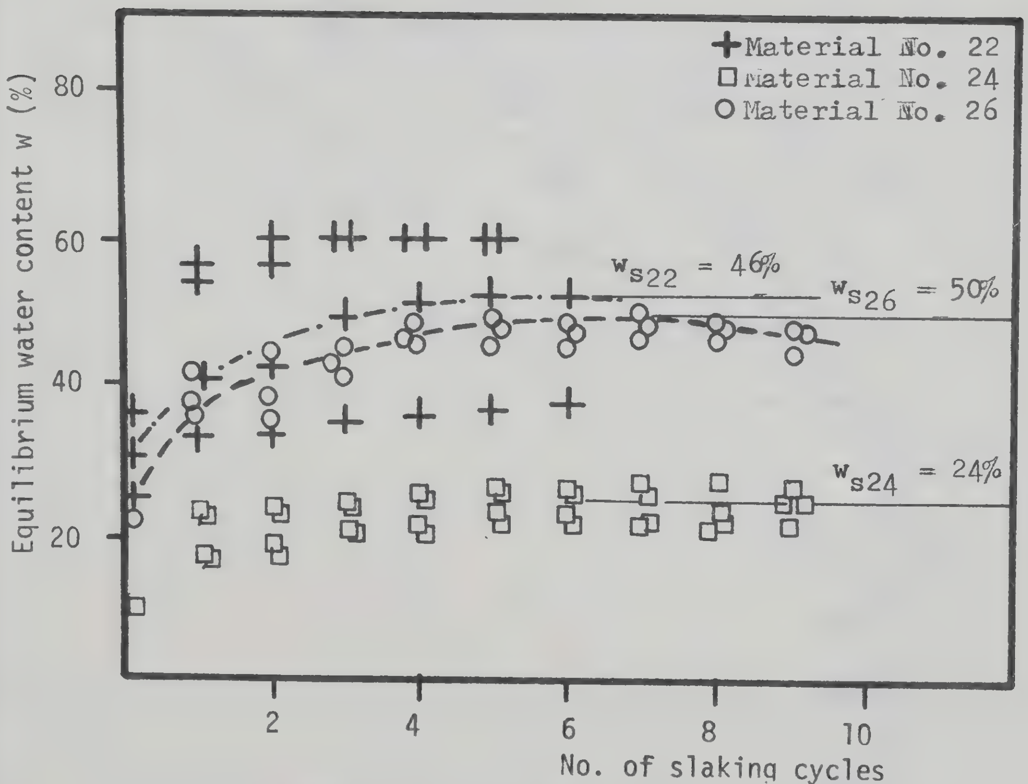


FIG. A 3.53. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE VERSUS NUMBER OF SLAKING CYCLES FOR MATERIALS NO. 22, 24 AND 26.

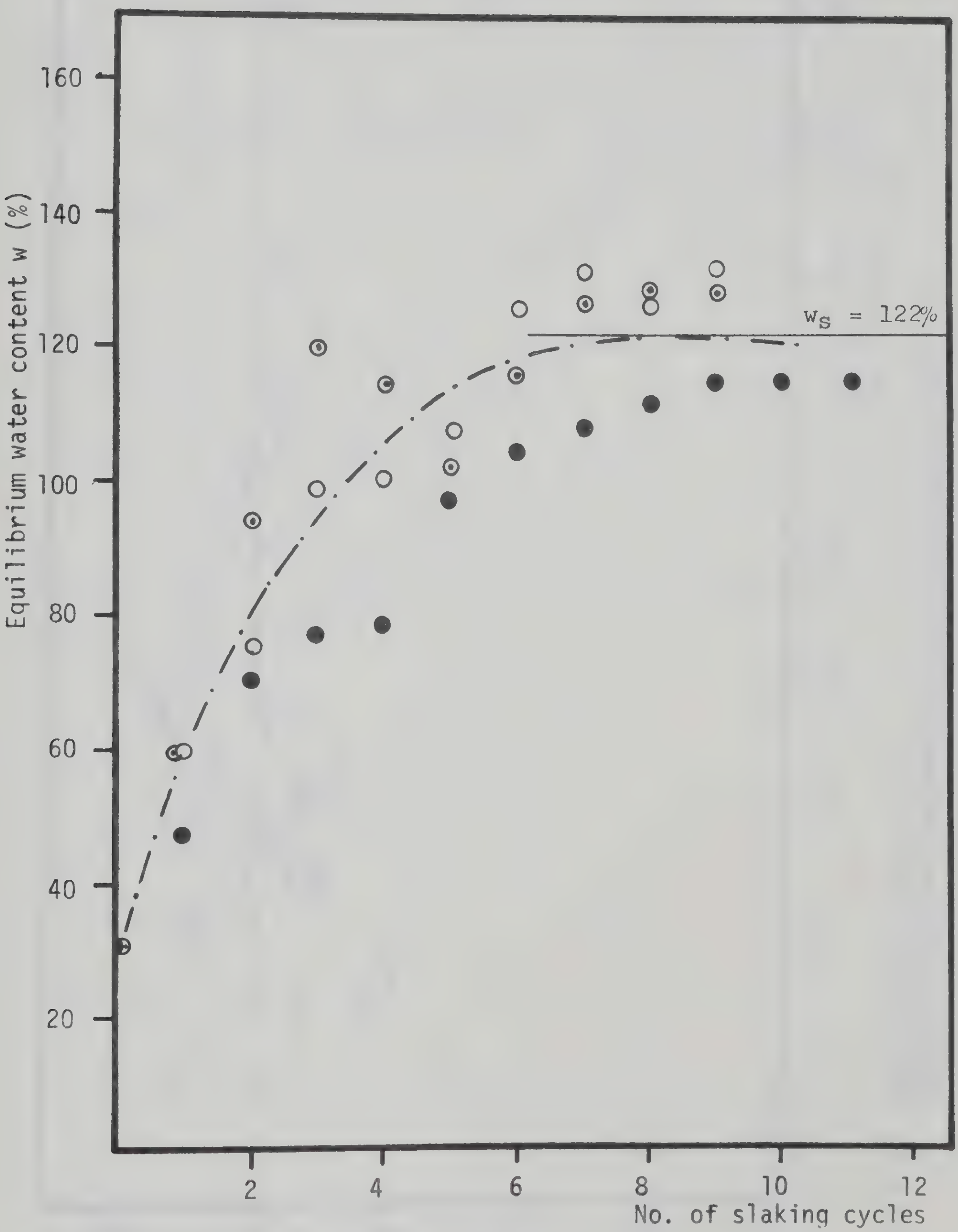
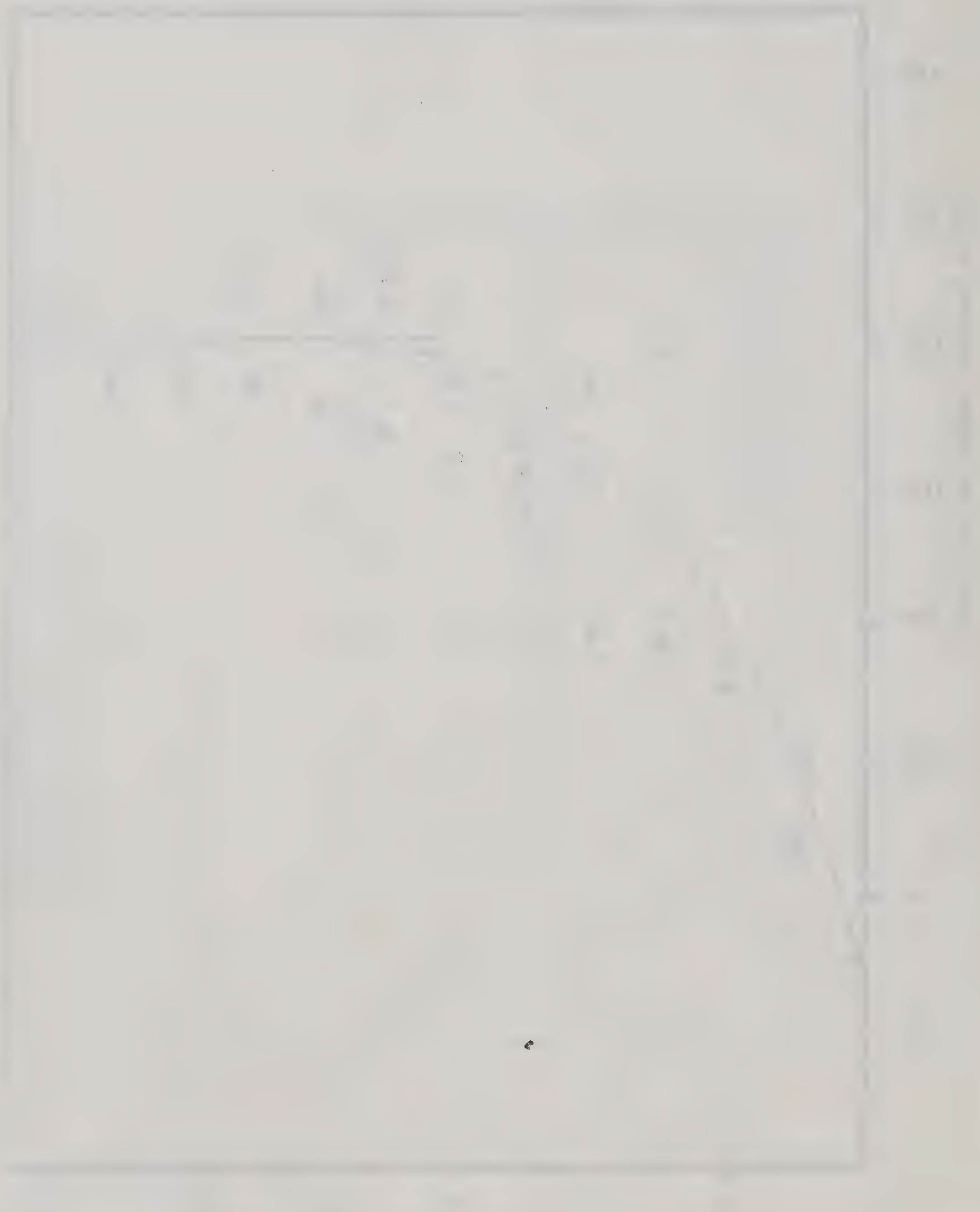


FIG. A 3.54. QUANTITATIVE SLAKING TEST:
STABILIZED WATER CONTENT AFTER EACH WETTING STAGE
VERSUS NUMBER OF SLAKING CYCLES FOR MATERIAL NO. 23.



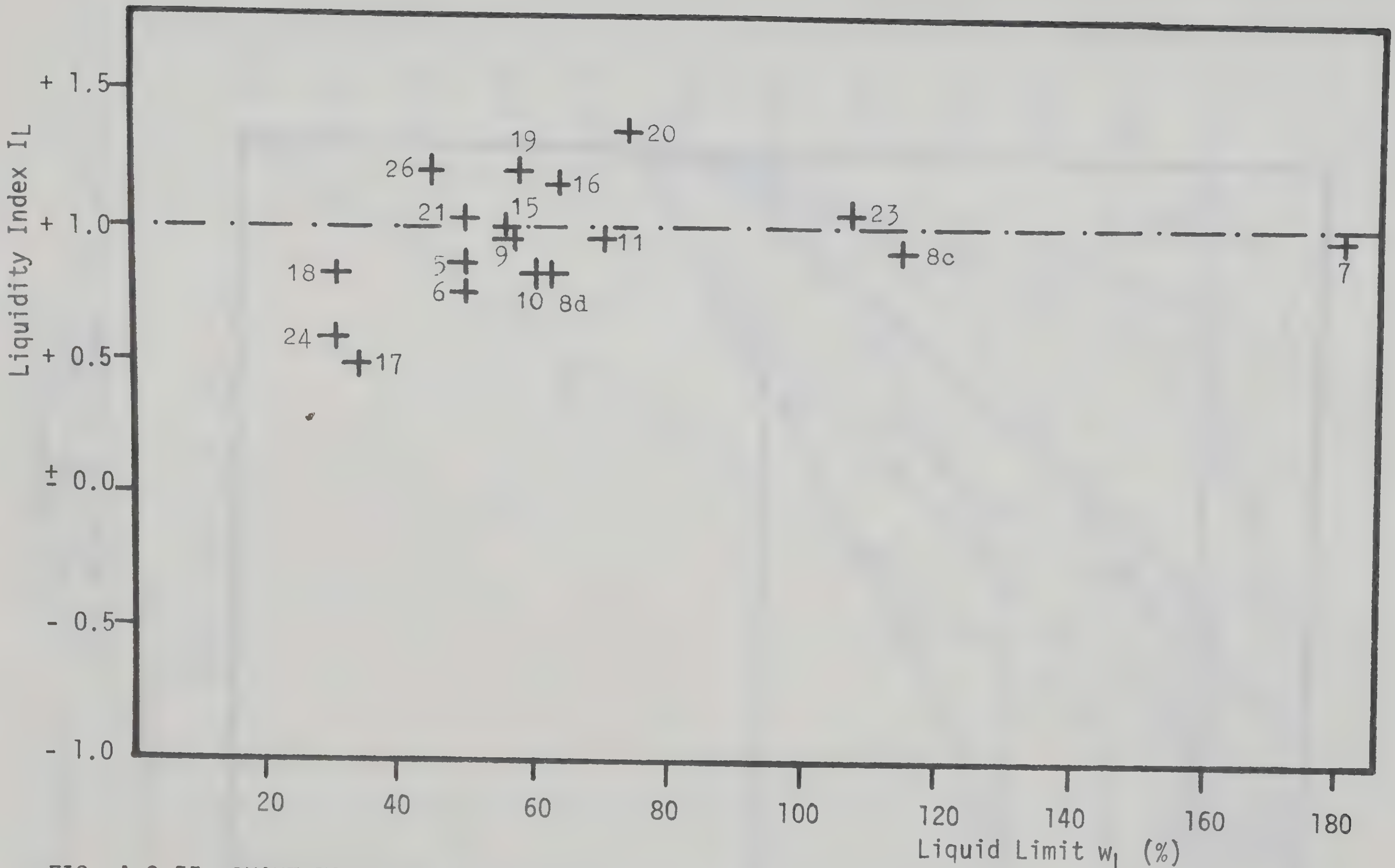
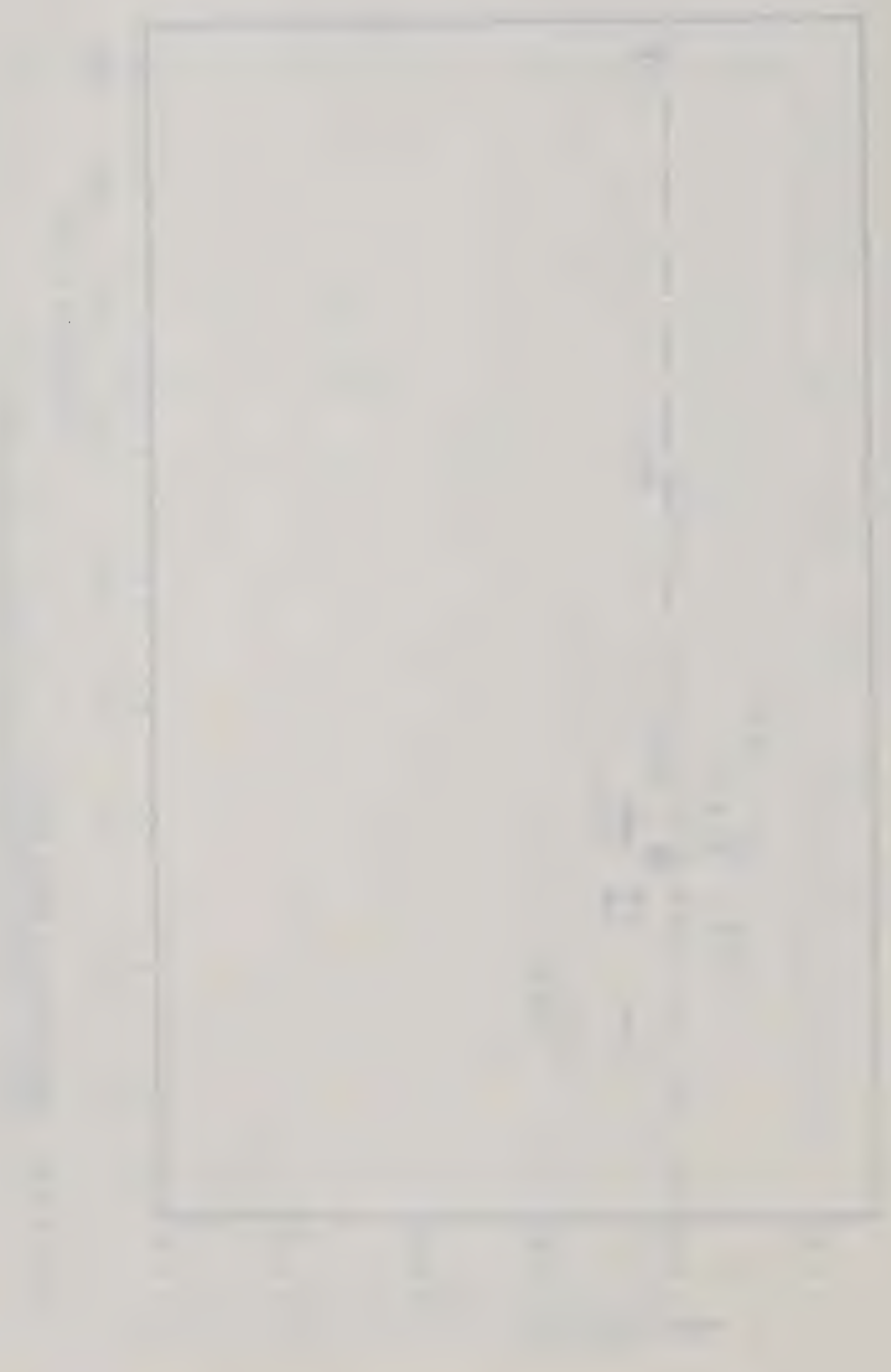


FIG. A 3.55. QUANTITATIVE SLAKING TEST:
LIQUIDITY INDEX AT MAXIMUM SLAKING VERSUS LIOUID LIMIT.



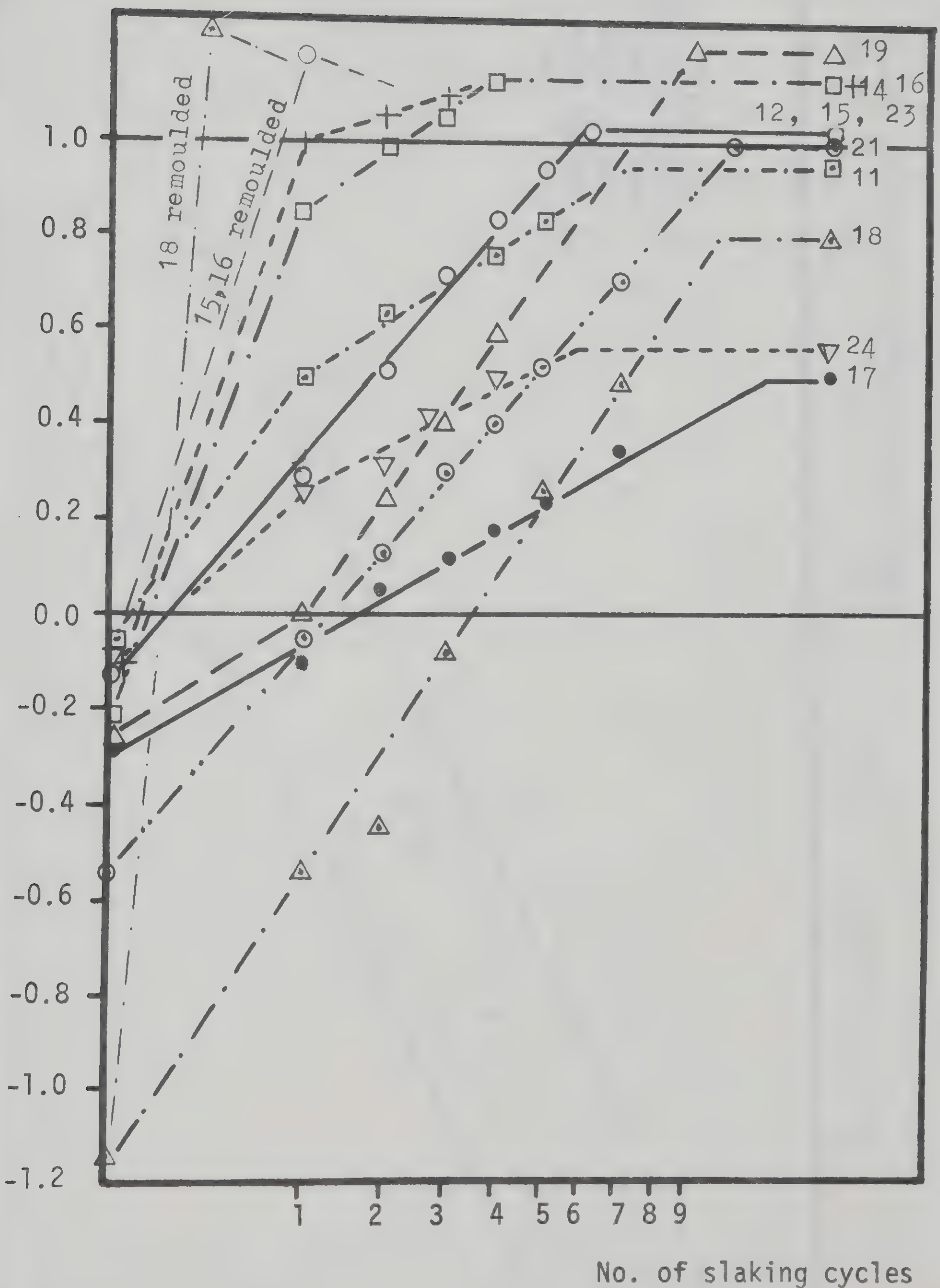


FIG. A 3.56. QUANTITATIVE SLAKING TEST:
LIQUIDITY INDEX VERSUS NO. OF SLAKING CYCLES
IN A SQUARE ROOT PLOT.

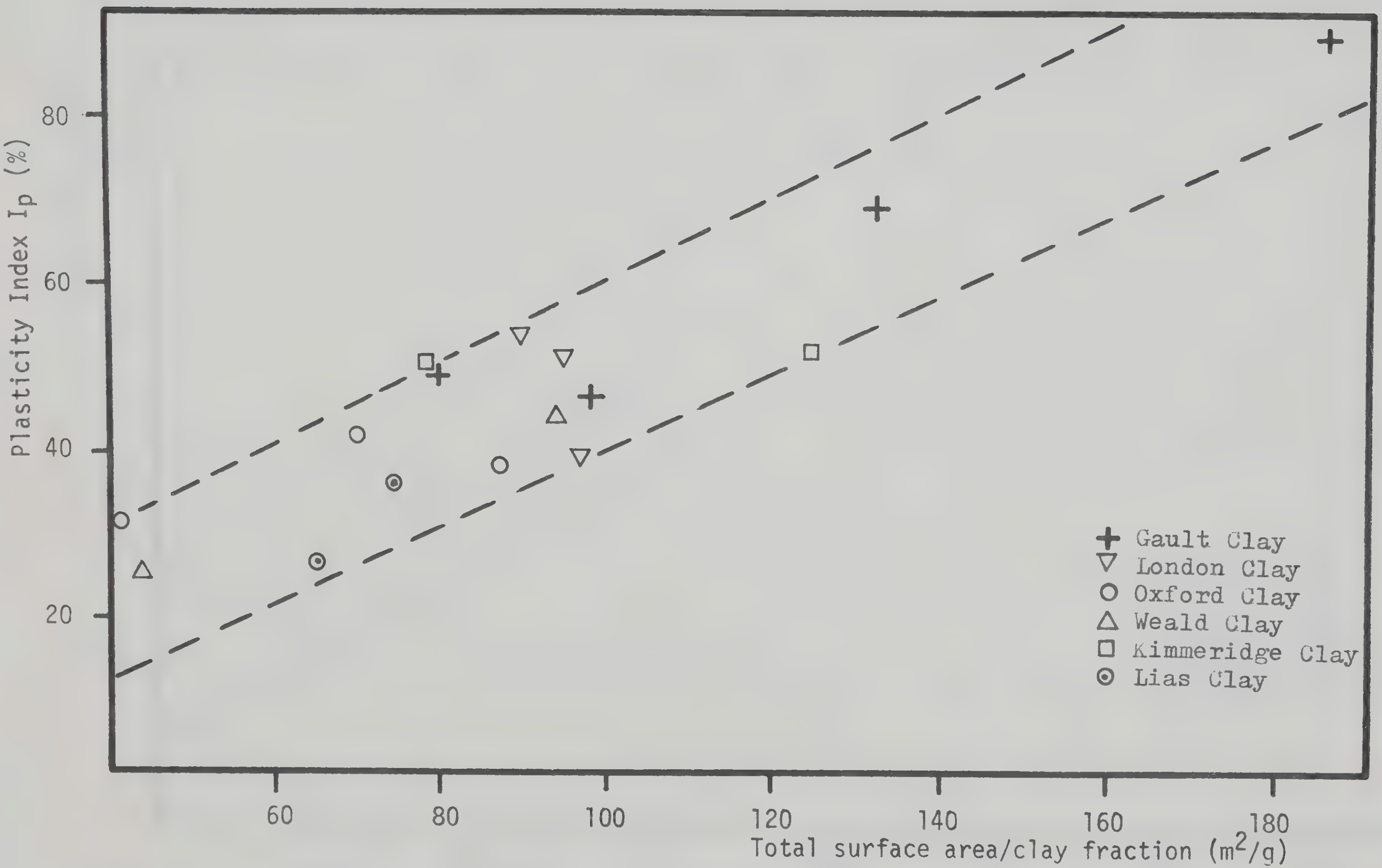


FIG. A 3.57. PLASTICITY INDEX VERSUS TOTAL SURFACE AREA OF CLAY FRACTION FOR ENGLISH CLAYS (AFTER FARRAR AND COLEMAN, 1967).

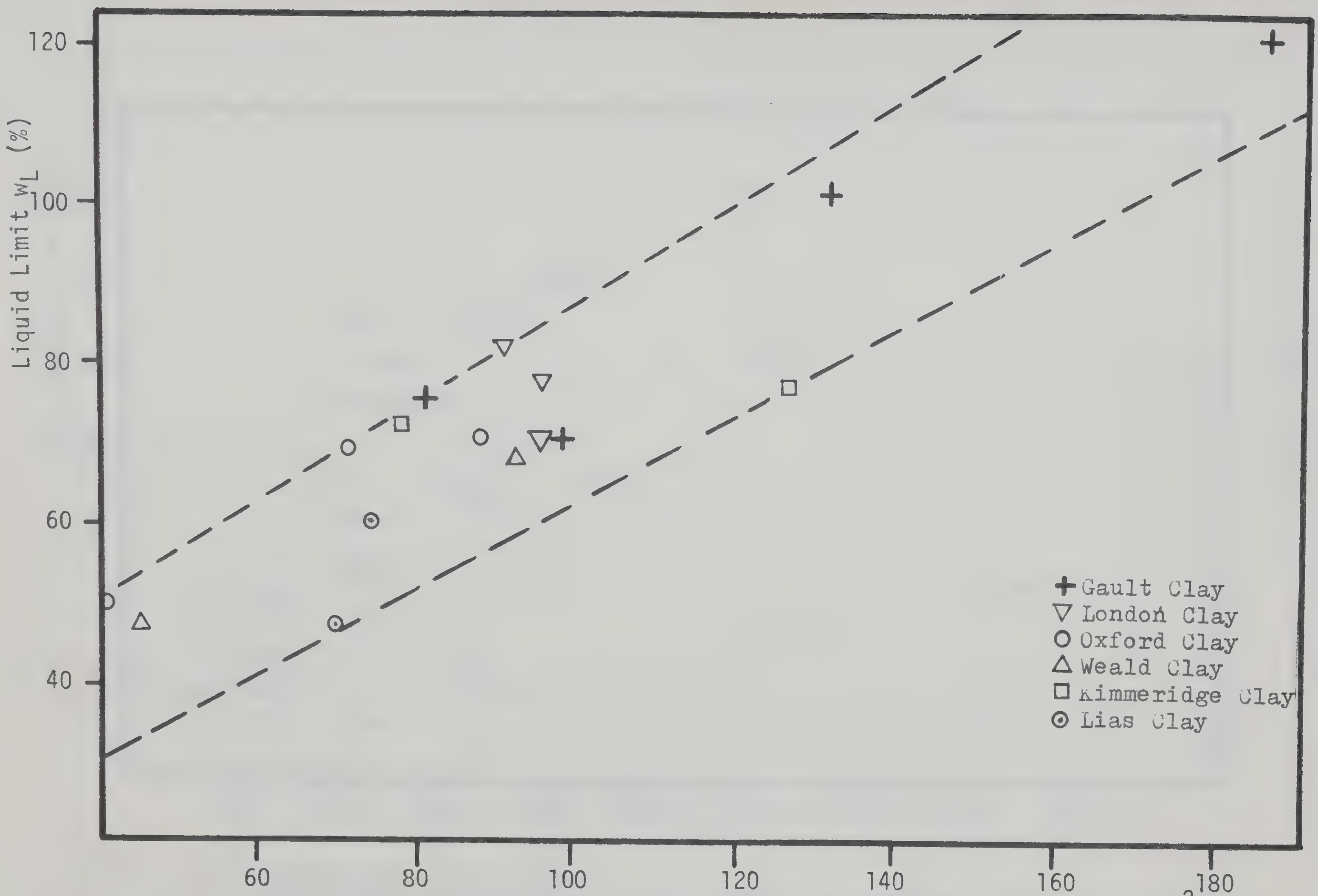


FIG. A 3.58. LIQUID LIMIT VERSUS TOTAL SURFACE AREA OF CLAY FRACTION FOR ENGLISH CLAYS (AFTER FARRAR AND COLEMAN, 1967).

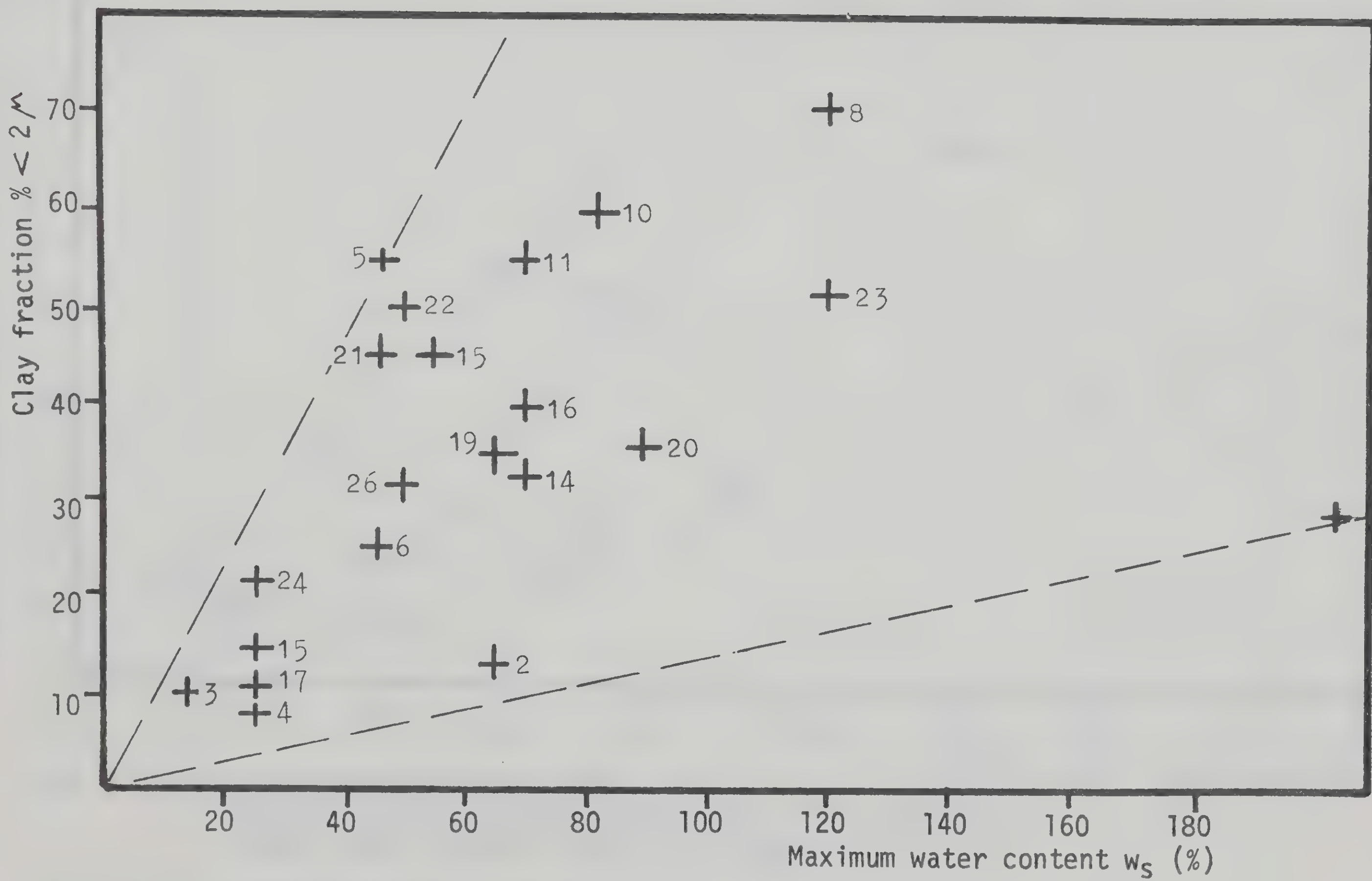


FIG. A 3.59. QUANTITATIVE SLAKING TEST:
CLAY FRACTION VERSUS WATER CONTENT AFTER FULL SLAKING.

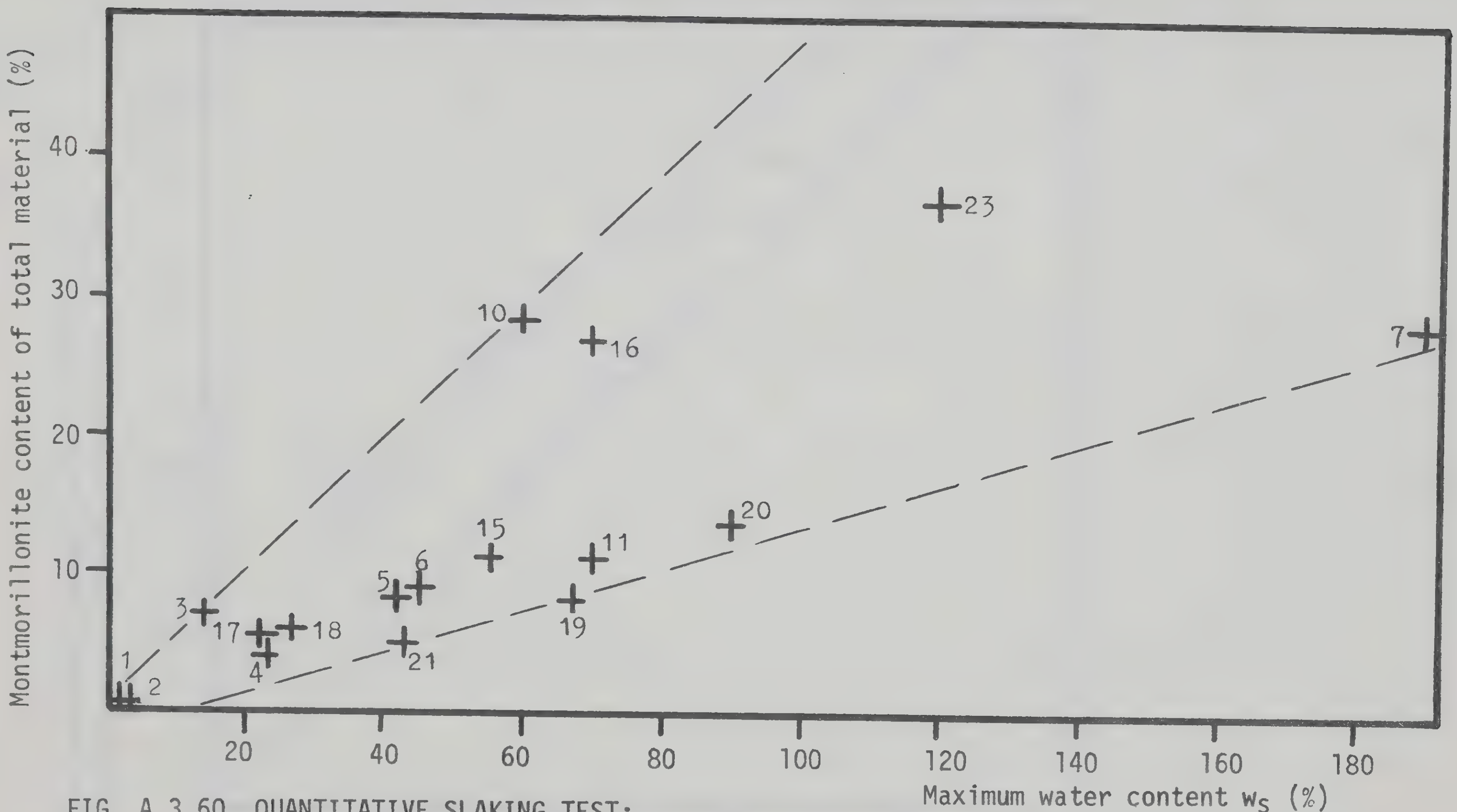


FIG. A 3.60. QUANTITATIVE SLAKING TEST:
MONTMORILLONITE CONTENT (REFERRED TO TOTAL MATERIAL)
VERSUS WATER CONTENT AFTER FULL SLAKING.

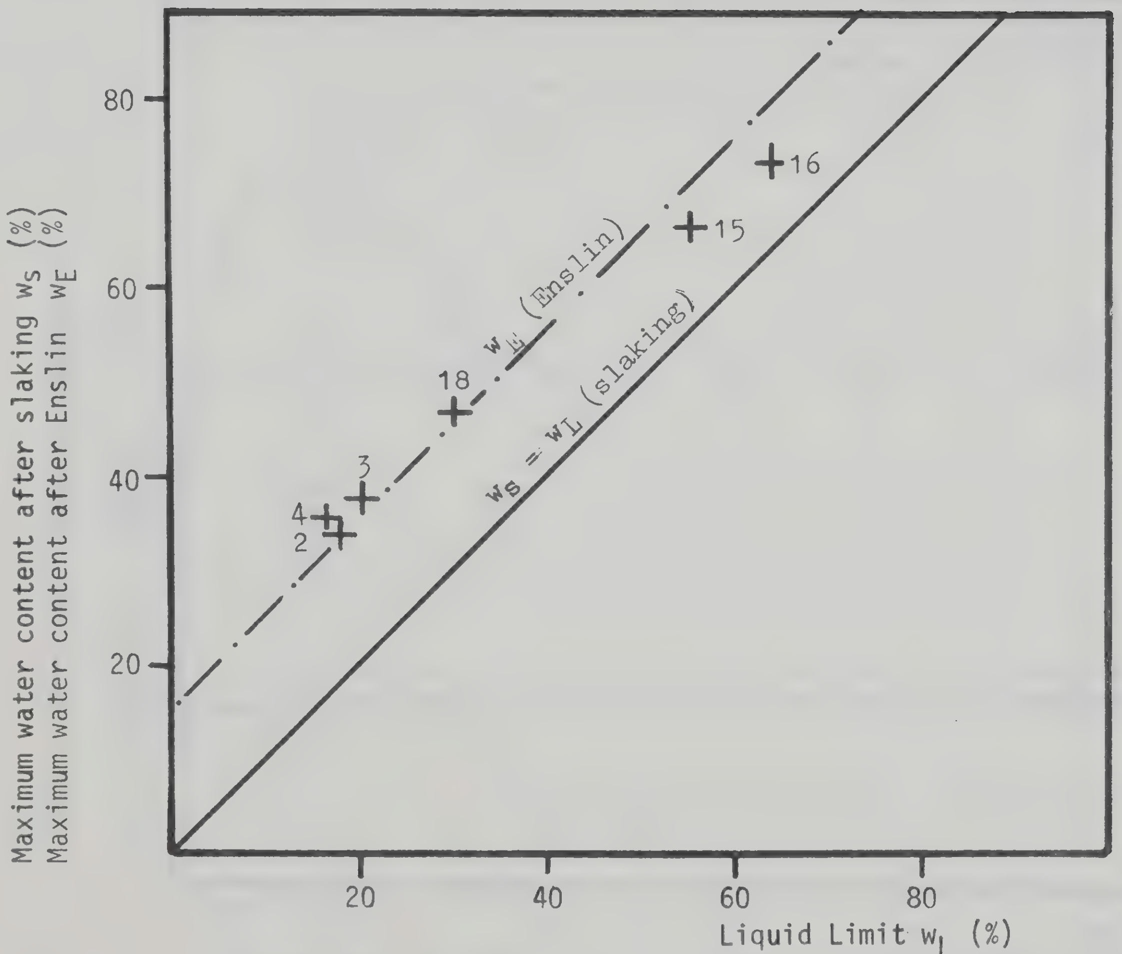


FIG. A 3.61. QUANTITATIVE SLAKING TEST:
COMPARISON OF MAXIMUM WATER CONTENT AFTER FULL SLAKING OF REMOULDED MATERIALS WITH WATER ABSORPTION AFTER ENSLIN (NEUMANN, 1957).

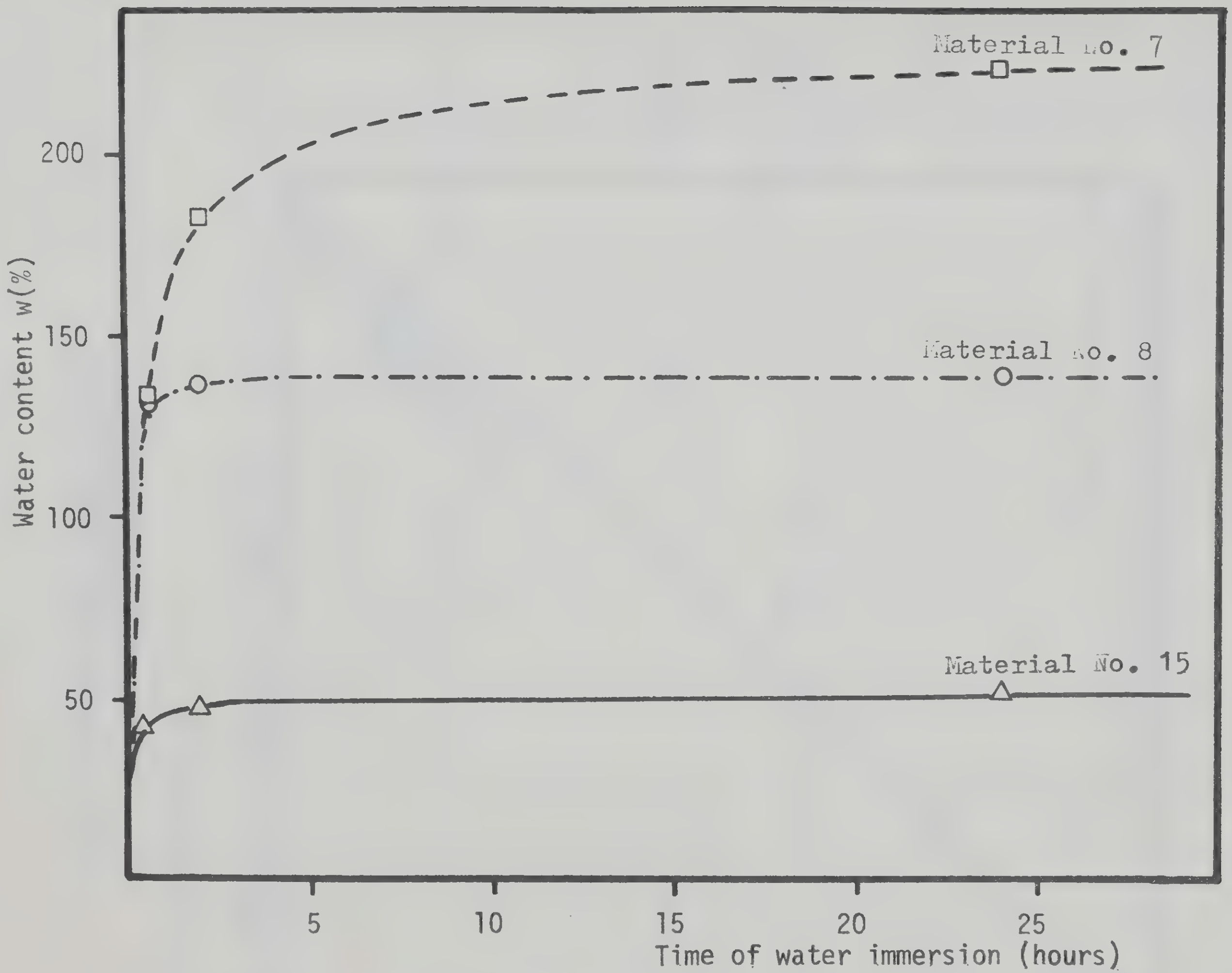


FIG. A 3.62. RATE OF SLAKING TEST:
INCREASE OF WATER CONTENT DURING WATER IMMERSION
FOR OVENDRIED SPECIMENS FOR TYPICAL MATERIALS.

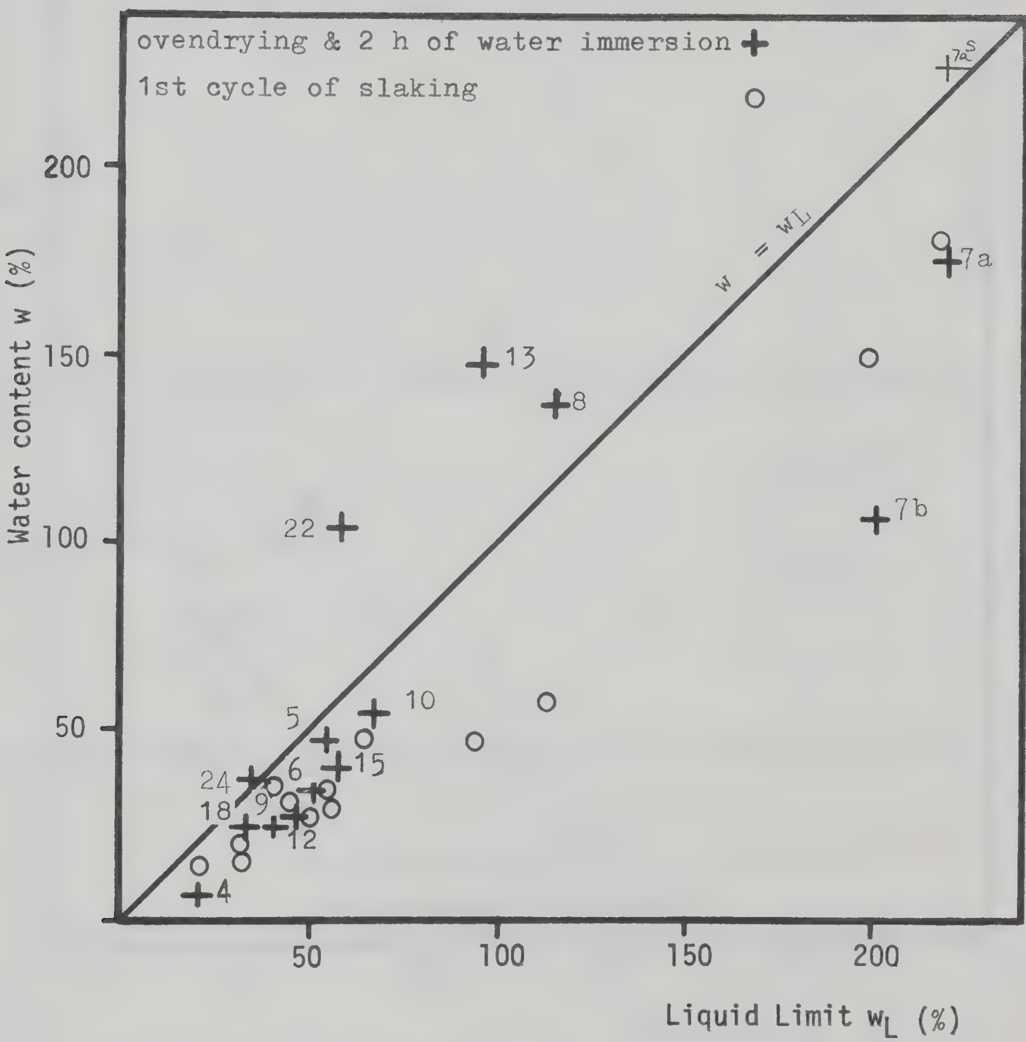


FIG. A 3.63. RATE OF SLAKING TEST:
WATER CONTENT VERSUS LIQUID LIMIT.

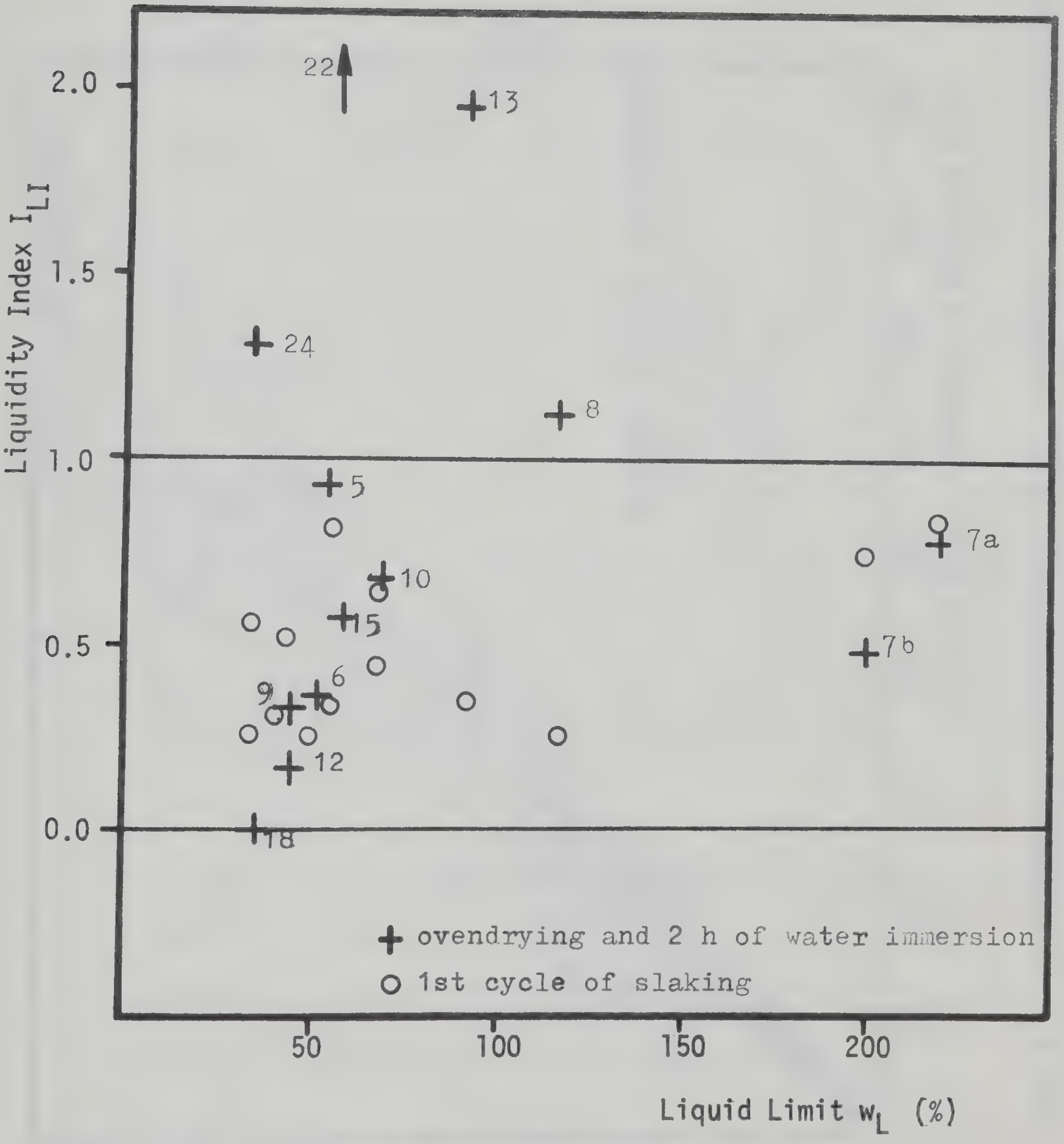


FIG. A 3.64. RATE OF SLAKING TEST:
LIQUIDITY INDEX VERSUS LIQUID LIMIT.

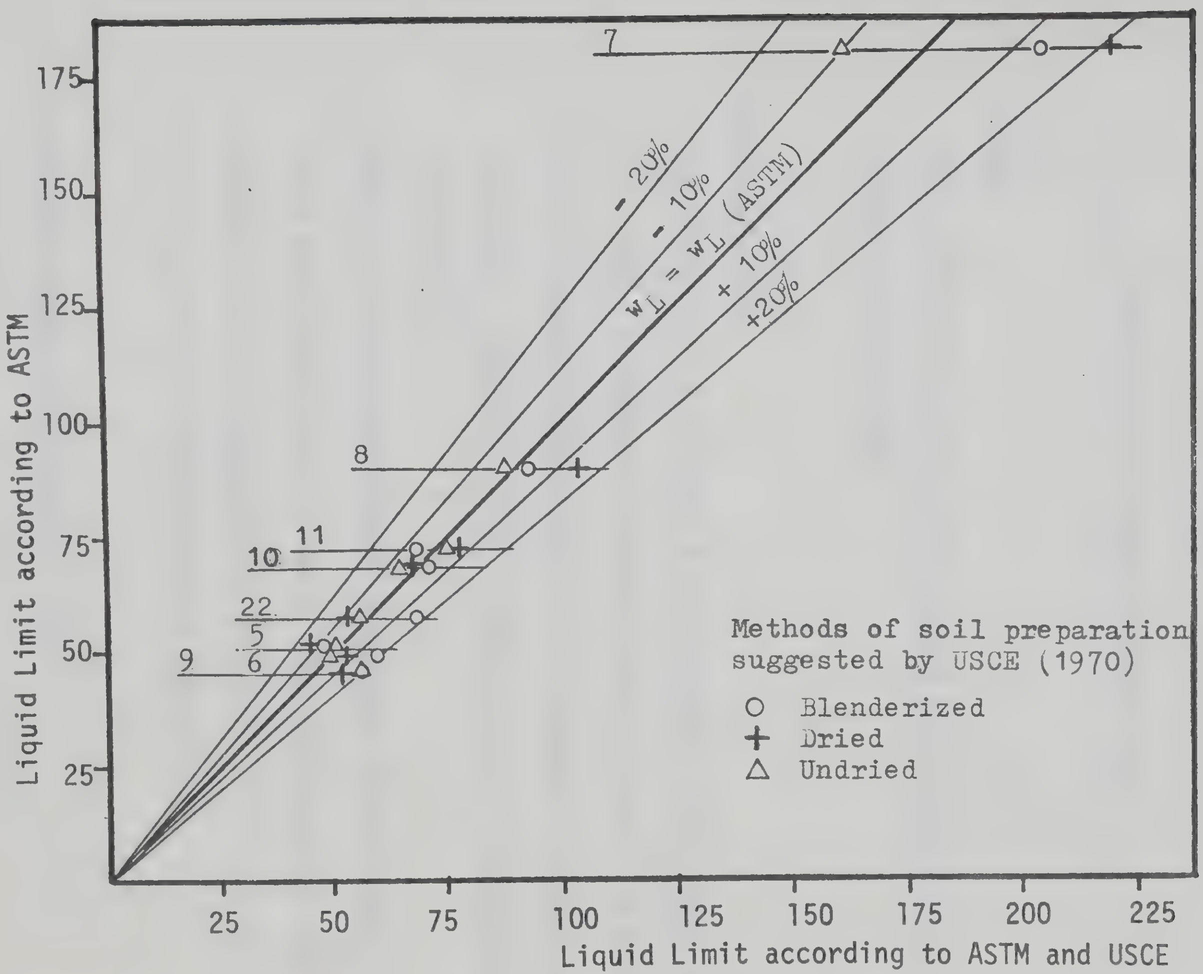


FIG. A 3.65. COMPARISON OF LIQUID LIMIT VALUES ACCORDING TO ASTM STANDARDS WITH VALUES ACCORDING TO USCE SUGGESTIONS.

APPENDIX B

PROCEDURE FOR CLASSIFICATION TESTS

I. Slaking

Find amount of slaking from Liquid Limit according to ASTM designation D423-54T.

II. Rate of Slaking

1. Take pieces of less than 1 inch diameter and ovendry them ($t = 105^{o}$ C). Determine natural water content, if possible.
2. Take dry weight of material to be immersed in water (around 20 grams), and place it into funnel which is covered with filterpaper.
3. Immerse the material, which is sitting in the funnel, into water and leave it for 2 hours. Tap water may be used.
4. Determine water content after 2 hours.
5. Find ΔI_L after 2 hours of water immersion.

III. Standard Compression Softening Test

Compression tests: (preferably σ_3 = 50 psi)

strain rate: 0.014"/min.

sample size: 6" to 2" diameter

1. Compression tests on 3 fresh specimens.

   If $c_{uo} > 300$ psi and rate of slaking is slow, no further tests are needed. In all other cases continue with 2.

2. Immerse 5 specimens of undisturbed materials into distilled water.

3. Do compression test on one sample after 1 hour:

   if $c_u/c_{uo} < 0.8$: do 3.1.

   if $c_u/c_{uo} > 0.8$: do 3.2.

   3.1. Do compression test on the 2nd and 3rd specimen immediately.

   if $c_u/c_{uo} < 0.5$: do 3.1.1.

   if $c_u/c_{uo} > 0.5$: do 3.1.2.

   3.1.1. Do compression test on the 4th and 5th specimen immediately.

   3.1.2. Do compression test on the 4th and 5th specimen after 4 hours.

   3.2. Do compression test on the 2nd and 3rd specimen after 2 days:

   if $c_u/ c_{uo} < 0.5$: do 3.2.1.

   if $c_u/c_{uo} > 0.5$: do 3.2.2.

3.2.1. Do compression test on the 4th and 5th specimen immediately.

3.2.2. Do compression test on the 4th and 5th specimen after 6 to 10 days.

Water contents should be determined after each compression test. Plot c_u/c_{uo} versus time of water immersion in a semilogarithmic plot and find time for 50% strength reduction t_{50}.